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# تقنيات معالجة مياه الشرب

# Drinking Water Treatment Technology

**November 2008**

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**CH / Mohamed Shaltout**
**Eng / Mohamed Ghonam**

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**Deutsche Gesellschaft für Technische Zusammenarbeit - GTZ**
**Water and Wastewater Management Programme**
**GTZ Project No. 06.2006.3**

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**ARAB REPUBLIC OF EGYPT**
**German Technical Cooperation**

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# Water treatment technology of Drinking Water

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# الفصل الأول

## مقدمة فى معالجة مياه الشرب

# الباب الاول

# مقدمة فى معالجة مياه الشرب

يرجع اهتمام الإنسان بنوعية الماء الذي يشربه إلى أكثر من خمسة آلاف عام . ونظرا للمعرفة المحدودة في تلك العصور بالأمراض ومسبباتها فقد كان الاهتمام محصور في لون المياه وطعمها ورائحتها فقط . وقد استخدمت لهذا الغرض . وبشكل محدود خلال فترات تاريخية متباعدة . بعض عمليات المعالجة مثل الغليان والترشيح والترسيب وإضافة بعض الأملاح ثم شهد القرنان الثامن والتاسع عشر الميلاديان الكثير من المحاولات الجادة في دول أوربا وروسيا للنهوض بتقنية معالجة المياه حيث أنشئت لأول مرة في التاريخ محطات لمعالجة المياه على مستوى المدن .

ففي عام ١٨٠٧م أنشئت محطة لمعالجة المياه في مدينة جلاسكو الأستكلندية ،وتعد هذه المحطة من أوائل المحطات في العالم وكانت تعالج فيها المياه بطريقة الترشيح ثم تنقل إلى المستهلكين عبر شبكة أنابيب خاصة . وعلى الرغم من أن تلك المساهمات تعد تطورا تقنيا في تلك الفترة إلا أن الاهتمام آنذاك كان منصبا على نواحي اللون والطعم والرائحة ، أو ما يسمى بالقابلية ، وكانت المعالجة باستخدام المرشحات الرملية المظهر السائد في تلك المحطات حتى بداية القرن العشرين . ومع التطور الشامل للعلوم والتقنية منذ بداية هذا القرن واكتشاف العلاقة بين مياه الشرب وبعض الأمراض السائدة فقد حدث تطور سريع في مجال تقنيات المعالجة حيث أضيفت العديد من العمليات التي تهدف بشكل عام إلى الوصول بالمياه إلى درجة عالية من النقاء ، بحيث تكون خالية من العكر وعديمة اللون والطعم والرائحة ومأمونة من النواحي الكيمائية والحيوية.

## معالجة المياه

لقد كان وباء الكوليرا من أوائل الأمراض التي اكتشفت ارتباطها الوثيق بتلوث مياه الشرب في المرحلة السابقة لتطور تقنيات معالجة المياه ، فعلى سبيل المثال أصيب حوالي ١٧٠٠٠ شخص من سكان مدينة هامبورج الألمانية بهذا الوباء خلال صيف ١٨٢٩م أدى إلى وفاة ما لا يقل عن نصف ذلك العدد . وقد ثبت بما لا يدع مجالا للشك أن المصدر الرئيس للوباء هو تلوث مصدر المياه لتلك المدينة . يعد التطهير باستخدام الكلور من أوائل العمليات التي استخدمت لمعالجة المياه بعد عملية الترشيح وذلك للقضاء على بعض الكائنات الدقيقة من بكتريا وفيروسات مما

أدى إلى الحد من انتشار العديد من الأمراض التي تنقلها المياه مثل الكوليرا وحمى التيفويد . وتشمل المعالجة ، ومن هذه العمليات ما يستخدم لإزالة عسر الماء مثل عمليات التيسير ، أو لإزالة العكر مثل عمليات الترويب .

ونظرا للتقدم الصناعي والتقني الذي يشهد هذا العصر وما تبعه من ازدياد سريع في معدلات استهلاك المياه الطبيعية ، النقية نوعا ما ، ونظرا لما يحدث من تلوث لبعض تلك المصادر نتيجة المخلفات الصناعية ومياه الصرف الصحي وبعض الحوادث البيئية الأخرى فإن عمليات المعالجة قد بدأت تأخذ مسارا جديدا يختلف في كثير من تطبيقات عن مسار المعالجة التقليدية.

وفي هذه المقالة سنستعرض بإيجاز طرق المعاجلة التقليدية لمياه الشرب **إضافة لبعض** الاتجاهات الحالية والمستقبلية لتقنيات المعالجة .

## طرق المعالجة التقليدية

تختلف عمليات معالجة مياه الشرب باختلاف مصادر تلك المياه ونوعيتها والمواصفات الموضوعة لها . ويجب الإشارة الى أن التغير المستمر لمواصفات المياه يؤدي أيضا في كثير من الأحيان إلى تغير في عمليات المعالجة . حيث أن المواصفات يتم تحديثها دوما نتيجة التغير المستمر للحد الأعلى لتركيز بعض محتويات المياه وإضافة محتويات جديدة إلى قائمة الموصفات . ويأتي ذلك نتيجة للعديد من العوامل مثل :

- التطور في تقنيات تحليل المياه وتقنيات المعالجة.
- اكتشاف محتويات جديدة لم تكن موجودة في المياه التقليدية أو كانت موجودة ولكن لم يتم الانتباه إلى وجودها أو مدى معرفة خطورتها في السابق.
- اكتشاف بعض المشكلات التي تسببها بعض المحتويات الموجودة أصلا في الماء أو التي نتجت عن بعض عمليات المعالجة التقليدية . هذا ويمكن تناول عمليات المعالجة التقليدية المستخدمة للمياه استنادا إلى مصادرها السطحية والجوفية

## معالجة المياه السطحية

تحتوي المياه السطحية ( المياه الجارية على السطح ) على نسبة قليلة من الأملاح مقارنة بالمياه الجوفية التي تحتوي على نسب عالية منها ، وهي بذلك بعد مياه يسرة ( غير عسرة ) حيث تهدف عمليات معالجتها بصورة عامة إلى إزالة المواد العالقة التي تسبب ارتفاعا في العكر وتغيرا في اللون والرائحة ، وعليه يمكن القول أن معظم طرق معالجة هذا النوع من المياه اقتصر على

عمليات الترسيب والترشيح والتطهير . وتتكون المواد العالقة من مواد عضوية وطينية ، كما يحتوي على بعض الكائنات الدقيقة مثل الطحالب والبكتيريا . ونظرا لصغر حجم هذه المكونات وكبر مساحتها السطحية مقارنة بوزنها فإنها تبقي معلقة في الماء ولا تترسب . إضافة إلى ذلك فإن خوصها السطحية والكيميائية باستخدام عمليات الترويب الطريقة الرئيسية لمعالجة المياه السطحية ، حيث تستخدم بعض المواد الكيمائية لتقوم بإخلال اتزان المواد العالقة وتهيئة الظروف الملائمة لترسيبها وإزالتها من أحواض الترسيب .ويتبع عملية الترسيب عملية ترشيح باستخدام مرشحات رملية لإزالة ما تبقى من الرواسب ، ومن المكروبات المشهورة كبريتات الألمنيوم وكلوريد الحد يديك ، وهناك بعض المكروبات المساعدة مثل بعض البوليمرات العضوية والبنتونايت والسليكا المنشطة. ويمكن أيضا استخدام الكربون المنشط لإزالة العديد من المركبات العضوية التي تسبب تغيرا في طعم ورائحة المياه. تتبع عمليتي الترسيب والترشيح عملية التطهير التي تسبق إرسال تلك المياه إلى المستهلك .

## معالجة المياه الجوفية

تعد مياه الآبار من أنقى مصادر المياه الطبيعية التي يعتمد عليها الكثير من سكان العالم . إلا أن بعض مياه الآبار وخصوصا العميقة منها قد تحتاج ألى عمليات معالجة متقدمة وباهظة التكاليف قد تخرج عن نطاق المعالجة هي إضافة الكلور لتطهير المياه ثم ضخها الى شبكة التوزيع ، إذ تعد عملية التطهير كعملية وحيدة لمعالجة مياه بعض الآبار النقية جدا والتي تفي بجميع مواصفات المياه ، الا أن هذه النوعية من المياه هي الأقل وجودا في الوقت الحاضر ، لذلك فأنه إضافة لعملية التطهير فان غالبية المياه الجوفية تحتاج الى معالجة فيزيائية وكيمائية إما لإزالة بعض الغازات الذائبة مثل ثاني أكسيد الكربون وكبريتيد الهيدروجين ، أو لإزالة بعض المعادن مثل الحديد والمغنيز والمعادن المسببة لعسر الماء، وتتم إزالة الغازات الذائبة باستخدام . عملية التهوية والتي تقوم أيضا بإزالة جزء من الحديد والمنجنيز عن طريق الأكسدة ، وقد يكون الغرض من التهوية مجرد كما يحدث لبعض مياه الآبار العميقة التي تكون حرارتها عالية مما يستدعي تبريدها حفاظا على كفاءة عمليات المعالجة الأخرى . أما إزالة معادن الحديد والمنغنيز فتتم بكفاءة في عمليات الأكسدة الكيمائية باستخدام الكلور أو برمنجنات البوتاسيوم .

ان الطابع العام لمعالجة المياه الجوفية هو إزالة العسر بطريقة الترسيب ، ويتكون عسر الماء بصورة رئيسة من مركبات الكالسيوم والماغنسيوم الذائبة في الماء . ويأتي الاهتمام بعسر الماء نتيجة لتأثيره السلبي على فاعلية الصابون ومواد التنظيف الأخرى ، بإضافة الى تكوين بعض

الرواسب في الغلايات وأنابيب نقل المياه. وفيما يلي استعراض موجز للعمليات المختلفة المياه الجوفية في هذا النوع من المحطات .

**أ . التيسير ( إزالة العسر ) بالترسيب**

تعني عملية التيسير أو إزالة العسر للمياه ( water softening) إزالة مركبات عنصري الكالسيوم والماغنسيوم المسببة للعسر عن طريق الترسيب الكيمائي . وتتم هذه العملية في محطات المياه بإضافة الجير المطفأ ( هيدروكسيد الكالسيوم ) إلى الماء بكميات محدودة حيث تحدث تفاعلات كيمائية معينة تتشكل عنها رواسب من كربونات الكالسيوم و هيدروكسيد الماغنسيوم . وقد يتم اللجوء في كثير من الأحيان الى إضافة رماد الصودا (كربونات الصوديوم ) مع الجير للتعامل مع بعض صور العسر . وتشمل عملية التيسير على حوض صغير الحجم نسبيا تتم فيه إضافة المواد الكيمائية حيث تخلط مع الماء الداخل خلطا سريعا لتوزيعها في الماء بانتظام ، ثم ينقل الماء الى حوض كبير الحجم ليبقي فيه زمنا كافيا لإكمال التفاعلات الكيمائية وتكوين الرواسب حيث يخلط الماء في هذه الحالة خلطا بطيا يكفي فقط لتجميع والتصادق حبيبات الرواسب وتهيئتها للترسيب في المرحلة التالية ,

**ب . الترسيب**

تعد عملية الترسيب من أوائل العمليات التي استخدامها الإنسان في معالجة المياه . وتستخدم هذه العملية لإزالة المواد العالقة والقابلة للترسيب أو لإزالة الرواسب الناتجة عن عمليات المعالجة الكيمائية مثل التيسير والترويب . وتعتمد المرسبات في أبسط صورها على فعل الجاذبية حيث تزال الرواسب تحت تأثير وزنها .تتكون المرسبات

غالبا من أحواض خرسانية دائرية أو مستطيلة الشكل تحتوي على مدخل ومخرج للماه يتم تصميميها بطريقة ملائمة لإزالة أكبر كمية ممكنة من الرواسب ، حيث تؤخذ في الاعتبار الخواص الهيدروليكية لحركة الماء داخل الخوض . ومن الملامح الرئيسة لحوض الترسيب احتوائه على نظام لجمع الرواسب ( الحمأة ) وجرفها إلى بيارة في قاع الحوض حيث يتم سحبها والتخلص منها بواسطة مضخات خاصة .

**ج . الموازنة ( إعادة الكربنة ):**

نظرا لأن المياه الناتجة هن عملية التيسير تكون في الغالب مشبعة برواسب كربونات الكالسيوم ، وحيث أن جزءا من هذه الرواسب يتبقى في الماء بعد مروره بأحواض الترسيب فإنه من المحتمل أن يترسب بعضها على المرشحات أو في شبكات التوزيع مما يؤدي إلى انسداد أو الحد من كفاءة المرشحات الشبكات . لذلك فإن عملية التيسير لضمان عدم حدوث تلك الأضرار . ومن عمليات الموازنة الأكثر استخداما في التطبيق التقليدية هي إضافة غاز ثاني أكسيد الكربون بكميات محددة بهدف تحويل ما تبقى من كربونات الكالسيوم الى صورة البيكربونات الذائبة .

**د . الترشيح :**

هو العملية التي يتم فيها إزالة المواد العالقة ( العكارة ) . وذلك بإمرار الماء خلال وسط مسامي مثل الرمل وهذه العملية تحدث بصوره طبيعية في طبقات الأرض عندما تتسرب مياه الأنهار الى باطن الأرض . لذلك تكون نسبة العكر قليلة جدا أو معدومة في المياه الجوفية مقارنة بالمياه السطحية ( الأنهار والبحيرات وأحواض تجميع مياه الأمطار ) التي تحتوي على نسب عالية من العكر . تستخدم عملية الترشيح أيضا في إزالة الرواسب المتبقية بعد عمليات الترسيب في عمليات المعالجة الكيمائية مثل الترسيب والترويب .

تعد إزالة المواد العالقة من مياه الشرب ضرورية لحماية الصحة العامة من ناحية ولمنع حدوث مشاكل تشغيلية في شبكة التوزيع من الناحية الأخرى . فقد تعمل هذه المواد على حماية الأحياء الدقيقة من أثر المادة المطهرة ، كما أنها قد تتفاعل كيمائيا مع المادة المطهرة كما أنها قد تتفاعل كيمائيا مع المادة المطهرة مما يقلل من نسبة فاعليتها على الأحياء الدقيقة ، وقد تترسب المواد العالقة في بعض أجزاء شبكة التوزيع مما قد يتسبب في نمو البكتريا وتغير رائحة المياه وطعمها ولونها.تتم عملية الترشيح داخل المرشح الذي يتكون من ثلاث أجزاء رئيسة وهي : صندوق المرشح والتصريف السفلي ووسط الترشيح . يمثل صندوق المرشح البناء الذي يحوي وسط الترشيح ونظام التصريف السفلي ، ويبني صندوق المرشح في العادة من الخرسانة المسلحة ، كما توجد في قاعة . الذي يتكون من أنابيب وقنوات مثقبة . طبقة من الحصى المدرج لمنع خروج حبيبات الرمل من خلال الثقوب . والغرض من نظام التصريف السفلي تجميع المياه المرشحة وتوزيع مياه الغسيل عند إجراء عملية الغسيل للمرشح . أما وسط الترشيح فهو عبارة عن طبقة من رمل السيليكون ، وحديثا أمكن الاستفادة من الفحم المجروش ورمل الجارنت . عند مرور المياه خلال وسط الترشيح تلتصق المواد العالقة في بجدران حبيبات الوسط ، ومع استمرار عملية الترشيح تضيق فجوات الوسط للمياه بحيث يصبح المرشح قليل الكفاءة وعند ذلك يجب

إيقاف عملية الترشيح وغسل المرشح لتنظيف الفجوات من الرواسب يتم في عملية الغسيل ضخ ماء نظيف بضغط عال من أسفل المرشح عبر نظام التصريف السفلي ينتج عنه تمدد الوسط وتحرك الحبيبات واصطدم بعضها مع البعض ، وبذلك يتم تنظيفها مما علق بها من رواسب . وتندفع هذه الرواسب مع مياه الغسيل التي تتجمع في قنوات خاصة موضوعة في أعلى صندوق المرشح ، وتنقل الى المكان الذي يتم فية معالجة مخلفات المحطة وتستمر عملية الغسيل هذه لفترة قصيرة من الزمن (٥ –١٠ دقائق) بعدها يكون المرشح جاهزا للعمل .

## هـ التطهير :

هو العملية المستخدمة لقتل الكائنات الحية الدقيقة المسببة للأمراض (الجراثيم )، وتتم هذه العملية باستخدام الحرارة ( التسخين ) أو الأشعة فوق البنفسجية أو المواد الكيميائية مثل البروم أو اليود أو الأوزون أو الكلور بتركيزات لا تضر بالإنسان أو الحيوان . وتعد طريقة التسخين الى درجة الغليان أولى الطرق المستخدمة في التطهير ولاتزال أفضلها في حمالات الطوارئ عندما تكون كمية المياه قليلة ، لكنها عير مناسبة عندما تكون كمية المياه كبيره كما في محطات المعالجة نظرا لارتفاع تكلفتها . أما استخدام الأشعة فوق البنفسجية والمعالجة بالبروم واليود فتعد طرقا مكلفة . هذا وقد انتشر استخدام الأوزون والكلور في تطهير مياه الشرب ، حيث راج استخدام الأوزون في أوربا والكلور في أمريكا . وفي الآونة الأخيرة اتجهت كثير من المحطات في الولايات المتحدة الأمريكية الى استخدام الأوزون بالرغم من عدم ثباته كيمائيا وارتفاع تكلفته مقارنة بالكلور، وذلك لظهور بعض الآثار السلبية الصحية لاستخدام الكلور ( الكلورة ) في تطهير مياه الشرب يتفاعل الكلور مع الماء مكونا حامض الهيبوكلوروز وأيونات الهيبوكلورايت ثم يتفاعل جزء من حامض الهيبوكلوروز مع الأمونيا الموجودة في الماء مكونا أمنيات الكلور ( الكلور المتحد المتبقي) ويطلق على ما تبقى من حامض الهيبوكلوروز وأيونات الهيبوكلورايت الكلور الحر المتبقي وهذه المركبات ( الكلور الحر والكلور المتحد )هي التي تقوم بتطهير الماء وقتل الجراثيم الموجودة به ، ولذلك تلجا كثير من محطات المعالجة الى إضافة الكلور بنسب تكفي للحصول على كلور حر متبقي يضمن تطهير الماء الخارج من المحطة بكفاءة عالية ، بل في الغالب تكون كمية الكلور المضاف كافية لتأمين كمية محدود من الكلور الحر المتقي في شبكة توزيع المياه ، وذلك لتطهير المياه من أي كائنات دقيقة قد تدخل في الشبكة .

# الفصل الثانى

## المياه الجوفيــة

# الباب الثانى

# المياه الجوفيـة

المياه الجوفية هى المياه التى تتجمع فى باطن الأرض من تسرب مياه الأمطار إليها ثم تظهر ثانية بشكل ينابيع أو عيون طبيعية أو يعمل على استخراجها بواسطة الآبار أو الطلمبات و مصدر المياه الجوفية هو مياه الأمطار التى تكون المياه السطحية التى يتسرب جزء منها فى باطن الأرض مكونا المياه تحت الأرضية ويغوص فى الأرض ويكون المياه الغائرة كما هو الحال فى وادى النيل والدلتا وذلك بمرور مياه النهر فى طبقات مسامية (permeable ) يسهل مرور المياه فيها ، ويستمر تسرب المياه حتى تصل الى الطبقة السفلى المشبعة بالمياه (saturated zone) ومستوى المياه فى هذه المنطقة يطلق علية المستوى المائى ويكون المنسوب الأعلى لهذه المياه ثابتاً ويعرف بالمنسوب الثابت للمياه الغائرة (Permanent W.T ) وينحدر هذا المنسوب فى اتجاه سير المياه . ويتحكم فى مياه هذه المنطقة التركيب الجيولوجى للطبقات ونوع المواد المكونة لهذا الطبقات . فعندما تجرى المياه فى طبقات مسامية ويسهل مرورها دون أن يعوقها عائق فإن هذه المياه تكون حرة فى جريانها ويطلق عليها اسم المياه الحرة ( Free Water ) .

أما إذا مرت المياه بين طبقتين يصعب مرور المياه بها فإنها تكون مقيدة بهذا الممر وتكون تحت ضغط بحكمة الطرف الأعلى للطبقة الطينية العليا وتسمى بالمياه المقيدة ( Confined Water ) هذا وتعلو الطبقة المشبعة بالماء منطقة قليلة التشبع ( Unsaturated ) وهى تتأثر عادة بمياه المرشح ومنسوبها الأعلى غير ثابت ويعرف بالمنسوب المتغير للمياه الغائرة فى الأرض ( Fluctuating Water Table )

## المياه الجوفية فى مصر

تعتبر المياه الجوفية مصدرا رئيسيا لكثير من موارد المياه العامة. ومصدر المياه الجوفية فى مصر هو مياه الأمطار التى تسقط فى بلاد الحبشة و السودان التى تكون نهر النيل . وعند سقوط الأمطار يتسرب جزء منها داخل الطبقات تحت السطحية (Subsurface ) مكوناً المياه جوفية. وهذه المياه تتكون من منطقتين الأولى طبقة غير مشبعة وهى تعلو الطبقة المشبعة ومياه هذه المنطقة غالبا ما تتأثر بالمياه السطحية والأنهار والقنوات ويتراوح مستواها بين ٦- ١٠

أمتار من سطح الأرض . وقد يصل هذا المستوى الى سطح الأرض فى أوقات الفيضان مسببا تكون برك فى الأماكن المنخفضة , ومنسوب المياه لهذه المنطقة غير ثابت ويسمى المنسوب المتغير أما الطبقة المشبعة فتمتد تحت الطبقة غير المشبعة وقد سبق الكلام عنها . ويمكن تقسيم المناطق من حيث الانتفاع بالمياه الغائرة فى الأرض الى أربع أقسام:-

١. وادى النيل والدلتا
٢. الصحراء الغربية
٣. الصحراء الشرقية وسيناء
٤. المنطقة الساحلية للبحر الأبيض المتوسط

**أولاً – وادى النيل والدلتا**

وهو عبارة عن مجرى متسع محفور فى الصخور الجيرية وهذا المجرى قد حفره النهر لنفسه فى الصخور فى العصور الماضية ثم ملأه برواسبه المتنوعة من الحصى والرمال والطين وغيرها , ويختلف اتساع هذا المجرى فى الأماكن المختلفة من الوادى وكذا يختلف عمقه .

ومصدر المياه التى تملأ مسام طبقات رواسبه هى من مياهه , وجزء من المياه التى تصل الى هذه الطبقات عن طريق وديان الصحارى المصرية كوادى قنا والوادى الأسيوطى وغيرهما أو عن طريق الحجر الرملى النوبى ( Nubian Sand Stone ) الذى يظهر على جانبى الوادى من حدود السودان الى جنوب إسنا والرواسب النهرية لوادى النيل والدلتا عبارة عن مستودع عظيم للمياه التى تصل إليها عن طريق النهر نفسه وهذه المياه تكون كتلاً هائلة متصلة. ومن ذلك يتضح أنه لا توجد فى وادى النيل والدلتا المياه المقيدة تقيدا مطلقا أى التى تنحصر بين طبقتين غير مسامتين التى ينشأ عنها الآبار الارتوازية التى ترتفع مياهها على سطح الأرض وتتدفق منها بسبب الضغط الواقع عليها . كما يتضح أن تسمية الآبار العميقة الموجودة بوادى النيل والدلتا بالمياه الارتوازية غير صحيحة بالمرة , ويجب عدم الاستمرار فى هذه التسمية والاستعاضة عنها بتعبير ( مياه عميقة تحت التربة ) كما يستعاض عن كلمة أبار ارتوازية بكلمة أبار عميقة .

**ثانيا الصحراء الغربية**

إن المصدر الرئيسي للمياه فى الصحراء الغربية هو الحجر الرملي النوبي , وهو مشبع بمياه الأمطار التى سقطت علية فى أواسط السودان حيث يظهر هناك على سطح الأرض . وبحكم انحدار هذا التكوين الجيولوجي الى اسفل كلما اتجهنا شمالا , كانت المياه بداخلة تحت ضغط الطبقات الجيرية التى تعلوه فى الصحراء الغربية , فإذا ما وجدت هذه المياه طريقها الى سطح الأرض ( على هيئة عيون أو أبار ارتوازية )

سالت وأصبحت موردا هاما لمياه الشرب والرى فى الواحات . وقد ذكرنا هنا كلمة أبار ارتوازية لأن الآبار التى تكون فى هذه المنطقة هى الوحيدة بهذه التسمية لأن **مياهها تكون** محصورة بين طبقتين وتخرج تحت ضغط . وهناك مصدران أخران للمياه فى هذه المنطقة , إحداهما النيل عند تقابلة مع الحجر الرملى النوبى الذى يسحب من ماء النيل حينئذ , والأخر مياه الأمطار على الساحل الشمالى غربى الدلتا – ومن أمثلة الأولى مياه أبار وادى النطرون والأديرة واستراحة طريق إسكندرية الصحراوى .

وقد لوحظ أن منسوب المياه فى الصحراء الغربية قد انخفض فى النصف القرن الماضى حوالى ١٠ امتار فى الواحة الداخلية وحوالى ٥ أمتار فى الواحة الخارجية ويعزى ذلك لكثرة عدد الأبار يكون فى واحتى الخارجة والداخلة , وأقلها فى واحة الفرافرة –أما فى واحتى البحرية وسيوة فهى بين وبين . أما نوع المياه فقد وجد أن اصلحها وأعذابها فى الواحات الخارجة والداخلة والبحرية ويلى ذلك المياه فى واحة الفرافرة –

**ثالثا الصحراء الشرقية**

مصدر الماء الرئيسى هو الأمطار , وهى تسقط على الجبال الشرقية المحاذية للبحر الأحمر وتسيل منها الى الأودية حيث تختزل فى الرمل الحصى.

منطقة سيناء – أهم مصدر للمياه فى هذة المنطقة هى الأمطار . وهى تتجمع فى الوديان مثل وادى العريش , فيران , الطور وقد ينتج من هذه المياه عيون , كما فى منطقتين ( قوس القسيمة والجديرات ) حيث تحفر بيارات لخزن المياه المنسابة منها . وقد اتخذت فى بعض المناطق التدابير اللازمة لخزن مياه السيول وذلك ببناء سدود مثل سد العريش , غير أنها وجد أن هذة السدود كثيرا ما تمتلىء بالرواسب الجروفة بالسيول أو تتحطم أمامها وتفضل البيارات ذات السدود الأرضية فى هذا الشق .

**رابعا – السهل الساحلى الشمالى**

ومصدر المياه فيها هو الأمطار التى تسقط على الكذبان الرملية , فتكون طبقة من المياه العذبة الطافية فوق المياه الراشحة من البحر . ويمكن الحصول على هذة المياه بعمل أبار ضحلة . وتحتوى المياه على قدر كبير من ( الكبريتات الذائبة ) وتزداد ملوحتها كلما اتجهنا غربا فى ساحل سيناء نحو بحيرة المنزلة – أما فى وادى العريش ورفح فهى أعذب من ذلك ولكنها بوجة عام أكثر ملوحة من نظيرتها فى الأبار الضحلة على السهل الساحلى غرب الاسكندرية. وهذا الأخير يحتوى على أبار قديمة ضحلة تسمى الأبار الرومانية يصل قاعها الى طبقة المياه العزبة الطافية ويحفر الأهالى هناك أبارا أخرى وبيارات لخزن مياه الأمطار ويوجد من هذة ما يقرب من الألف بيارة بين الاسكندرية والسلوم .

## المناطق التى يتعذر الحصول فيها على مياه صالحة للشرب فى وادى النيل

هناك مناطق فى دلتا النيل لا يتيسرالحصول فيها على مياه جوفية صالحة للشرب نظرا لزيادة الأملاح الذائبة بها وهذة المناطق هى :

١ منطقة الفيوم وذلك لوجود طبقات الصخور الجيرية وهى ذات سمك كبير مما يتعذر معة دق أبار عميقة للوصول الى المياه الغائرة كما أن طبقات الرمل والطين التى تعلو هذة الطبقة الجيرية لا يزيد سمكها على بضعة أمتار وهى تحتوى فقط على مياه الصرف المالحة ولذا لا توجد آبار مياه صالحة.

٢ منطقة شمال الداتا الصحراوية :- وذلك لقربها من البحر الأبيض المتوسط والبحيرات الشمالية وفى هذة المنطقة نجد أن مصدر مياه الشرب هو مياه الأمطار التى تكون طبقة رفيعة لا يزيد سمكها على متر واحد تطفو على المياه المالحة التى تشبع باطن الأرض فى هذة المنطقة .

٣ منطقة قناة السويس وذلك لقربها من قناة السويس والبحيرات المرة وتحتوى طبقاتها على مياه قد تزيد ملوحتها على ملوحة مياه البحر بسبب التبخر وتركيز الأملاح بها

٤ بعض المناطق فى وادى النيل التى تتاخم الصحراء كمناطق الشرقية والبحيرة وبعض المناطق المتفرقة فى الوجة القبلى .

## التكوين الطبيعى للمياه الجوفية

لطبيعة تكوين الارض فى أى جهة أثرا كبيرا ليس فى كمية الماء الجوفية التى يمكن الحصول عليها فحسب بل فى نوع هذة المياه وخواصها الكيماوية والبكتريولوجية . ولتفسير ذلك نذكر أن مياه الأمطار عندما تسقط على سطح الارض فانها تحمل بعض المواد التى تسبب عكارتها ( Turbidity ) كما تلتقط بعض المواد التحللة ( Decomposed ) التى تحوى عددا كبيرا من البكتريا . وعندما تغوص هذة المياه السطحية فى باطن الارض تقوم طبقات الارض الرملية بحجز المواد العالقة بالمياه ومن ضمنها البكتريا . ولذلك فان المياه المستخرجة من الآبار العميقة تكون غالبا رائقة خالية من العكارة . وهذة العملية الطبيعية للترشيح لها أثر كبير فى حجز البكتريا والتقليل منها ويقل عدد من البكتريا كلما زاد تسرب المياه فى الأرض فى طبقات رملية . وقد قام كابر هل ( Cabrhel ) سنة ١٩٠٦ بعمل تحاليل وجد منها أن عدد البكتريا فى السنتيمتر المكعب من المياه السطحية قد بلغ عدة ملايين , ووصل هذا العدد الى بضع مئات على عمق نصف متر من سطح الارض ، ومائة فى كثير من العينات التى أخذت على عمق زيادة عن متر . أما الآبار العميقة فقد وجد أن مياهها تحوى عددا قليلا من البكتريا العادية ( أقل من ١٠٠ ) وهذه الابحاث تؤيدها التحاليل التى يقوم بها قسم المياه بوزارة الصحة لعينات المياه المختلفة التى ترد له يوميا من أنحاء الجمهورية .

أما من الوجهة الكيماوية فاننا نجد أن المياه الجوفية تختلف فى طبيعتها عن المياه السطحية . فنظرا لأن تربة الارض وصخورها تحوى أملاحا قابلة للذوبان ، فان المياه تذيب أثناء تسربها فى باطن الارض نسبة كبيرة من هذة الاملاح مما يسبب زيادتها فى المياه الجوفية عنها فى المياه السطحية . فبينما نجد أن الأملاح الذائبة فى مياه النيل تتراوح من ١٥٠ – ٢٥٠ جزء فى المليون نجد أن الاملاح فى الآبار الجيدة تبلغ حوالى ٢٥٠ الى ٥٠٠ جزء فى المليون وفى بعض الآبار أقل جودة من ٥٠٠ الى ٨٠٠ جزء فى المليون وفى الباقى من ٨٠٠ – ١٠٠٠ وهذا هو الحد الأقصى الذى تسمح به وزارة الصحة فى المياه الصالحة للشرب واذا زادت الاملاح عن ١٠٠٠ يعتبر غير صالح .

ويمكن أن نلخص خواص المياه الجوفية فى الأتى :-

١ - أنها رائقة نتيجة لعملية الترشيح التى تحدث لها أثناء تسربها فى باطن الارض . وقد يجدث أن تكون المياه المستخرجة من البئر رائقة ولكنها تتعكر بعد أستخراجها بفترة بسيطة وهذا يرجع الى ترسب الحديد والمنجنيز وكربونات الكالسيوم والمغنسيوم نتيجة فقدان الغازات التى تجعل هذة المواد فى حالة ذائبة ( عادة ثانى أكسيد الكربون )

٢ - الرائحة – ليس للمياه الجوفية فى العادة رائحة , ولكن بعضها قد يكون محملا بالهيدروجين المكبرت ويكسب المياه رائحة وطعما خاصين , كما أن الآبار الملوثة تكون للمياه المستخرجة منها رائحة كريهة وغير مستحبة .

٣ - الطعم – يتأثر طعم المياه الجوفية بما يكون زائدا فيها من الغازات والأملاح فالمياه التى بها نسبة عالية من الحديد والمنجنيز يكون لها طعما قابضا. كم أن المياه ذات النسبة العالية من الأملاح خاصة كلوريد الصوديوم يمكن تمييز طعمها بالمذاق . والى جانب ذلك فان عسر الماء يمكن تميزة بالمذاق خاصا اذا كانت كمية أملاح الماغنسيوم كبيرة

٤ - المياه الجوفية غالبا باردة فى الصيف دافئة فى الشتاء .

٥ - والى جانب ذلك قد تحوى المياه الجوفية على مواد عضوية ووجود هذة المواد العضوية فى المياه أمر غير مرغوب فية لأنة دليل على وجود مصدر للتلوث – والمواد العضوية التى توجد فى التربة الأرضية والتى تتسرب مع المياه الأرضية على نوعين أساسين ، كربونية وأزوتيه , والمواد العضوية الكربونية , يمكن أن تتأكسد الى ثانى أكسيد الكربون أمام المواد العضوية الأزوتيه فتتأكسد الى نتريت ونترات .

## تلوث المياه الجوفية

مما لاشك فية أن المياه الجوفية عرضة للتلوث . ويختلف مدى هذا التلوث حسب نوع المصدر وحسب حالة البئر ونوع طبقات الأرض . كما أن مدى تسرب التلوث فى باطن الأرض انما يتوقف على نوعة وقد ثارت عدة أراء حول مدى تأثر البئر بمصادر التلوث القريبة منة وأختلفت الآراء حول مدى تسرب هذا التلوث . واننا نرى أن لكل مورد ظروفة الخاصة حسب حالتة وحسب طبيعة المصدر وحسب طبقات الأرض كما ذكرنا وهذة كلها عوامل يجب أن تؤخذ فى الحسبان عند تقرير صلاحية المورد أو عدم صلاحيتة . فقد يظهر الفحص الكيماوى والبكتريولوجى لمرود ما , عدم صلاحيتة لوجود ما يشير الى تلوث المياه فى حين يكون البئر بعيدا كل البعد عن مصادر التلوث ثم يتضح ذلك فعلا فى كثير من عمليات المياه الجوفية التى عوينت بمعرفة قسم المياه نظرا لتكرار عدم صلاحية العينات المأخوذة منها .

ويمكن أن نذكر بصفة عامة أن التلوث البكتيري يتسرب إلي مسافة حوالي ١.٥ متراً والتلوث الكيماوي لمسافة ٢.٥متراً من قاع يؤرة التلوث كالمجارير الغير صماء وغيرها بشرط أن يكون

ذلك التسرب تحت منسوب مياه الرشح في تربة ناعمة مغمورة دائماً بالمجاري السائلة . كما أن البكتريا تتسرب مع تيار التلوث السائل لاجانبياً ولا ضده . كما أن سير التلوث يزداد في حالة الأرض ذات الحبيبات الكبيرة نوعاً .

ويمكن القول أن التلوث سواء أكان بكتيريا أو كيماوياً يتسرب مع تيار المياه في الارض إلي أسفل ولكنه ينحرف في اتجاهه عند مقابلته تيار المياه الجوفية. وإذا ما قرب منسوب مياه الرشح من سطح الارض تعرضت المياه للتلوث ببكتريا القولون كما أن قدرة التربة علي حجز البكتريا تتوقف علي حجم الحبيبات فاذا كبرت حجماً زاد تغلغل البكتريا فيها والعكس كما أن البكتريا تمكث مدة أطول في الأرض الخشنة عنها في الأرض الناعمة بعد وصولها اليها , وحينئذ قد تتكاثر تكاثراً يفوق في أثره مدي التلوث السطحي الأول .

أما عن مدي سيري التلوث مع تيار المياه الجوفية فانه بناء علي عدة مشاهدات نذكر أن البكتريا القولونية تتسرب مع المياه الجوفية في اتجاهها الي مسافة ٧ متر قدماً أما تتسرب المواد الكمياوية الذاتية ( النوشادر والازوتيت والازوتات ) فيكون الي مدي أطول وبسرعة أكبر وتدل التقارير علي امكان تسربها الي مسافة تتراوح بين بضعة أقدام الي عدة أميال . وتشير بعض التجارب التي أجريت أن سير التلوث البكتيري لا يزيد عن ٥ أمتار أما التلوث الكيميائي فقد يصل الي ١٠٠ متر .

## تلوث المياه الجوفية في وادي النيل والدلتا :

من المعلوم أن الرواسب النيليه تتكون من طبقات مسامية غير منتظمة شأن جميع الرواسب النهرية وتحمل هذه الطبقات المياه الغائرة التي تسير في مجموعها من الجنوب الي الشمال في طريقها الي البحر وهو المنفذ الطبيعي لها ويمكن تلخيص أنواع هذه الطبقات فيما يلي:-

١ -الطبقة العليا تتكون من رواسب نهرية بسمك حوالي ٦ الي ١٠ متراً يمكن للمياه أن تتسرب خلالها , وهي تختلف في طبيعتها من طبقات طينيه غير مسامية الي طينيه رملية .

٢ -يلي هذه الطبقة طبقة أخري من الرمل والرمل الطيني المشبع بالماء بسمك حوالي ١٠الي ١٥ متر . وهي طبقة ذات مسام دقيقة جداً بحيث يصعب سحب المياه منها بالطلمبات بكميات وافره وهي ذات مقدرة كبيرة علي حجز وترشيح البكتريا.

٣ ويلي الطبقة الثانية طبقة ثالثة من الرمل الخشن والزلط التي يمكن سحب المياه بسهولة بواسطة الأبار العميقة .

٤ ومن المعلوم أن منسوب المياه الجوفية علي عمق حوالي ١٠ متراً ويتغير هذا المنسوب صعوداً وهبوطاً تبعاً لمنسوب مياه النيل وقت الفيضان والتحاريق , وبذلك تنشأ عمليتي ترشيح وتجفيف علي نطاق واسع واذا أمكن حماية الأبار في وادي النيل والدلتا من التلوث السطحي أمتنع تلوثها من مصادر التلوث كالمجارير التي يبلغ عمقها حوالي ١٠ متراً وذلك اذا بلغت هذه الأبار عمقاً يتراوح بين ١٣ و ١٥ متراً أي وصلت الي الطبقة المشبعة بالمياه وتحت منسوبها البكتريا اليها وخاصة مع قصر مدة التخزين في طبقات الأرض الذي هو عامل من عوامل اهلاك البكتريا .

وبناءً علي ذلك فانه لضمان عدم تلوث الأبار يجب أن يكون سحب المياه من أعماق تزيد عن العشرة أمتار وهو متوسط عمق المنسوب للمياه الجوفية .

وهناك عوامل أخري يجب أن ندخلها في اعتبارنا عند دراسة احتمال التلوث كنوع المصدر وعمقه وتصميم البئر ومسامية الأرض ومدي انخفاض مخروط السحب وتأثيره علي المنسوب المائي

## الطريقة الصحيحة للحصول علي المياه الجوفية

أن الطريقة الصناعية الوحيدة للحصول علي المياه الجوفية هي بواسطة الأبار وهناك اشتراطات يجب أن تتوافر فيه مكان البئر وفي اقامته وتعقيمه وحمايته من التلوث .

وكل مال أو جهد يبذلان في اقامة بئر دون مراعاة الشروط الصحية إنما هو مال ضائع وجهداً بلا نتيجة وهذا يستلزم تعاون المختصين الصحيين والكيماويين والمهندسين الصحيين وغيرهم .

## إختيار موقع البئر

من أهم مايجب مراعاته عند إقامة بئر جديد إختيار موقعه بحيث يكون هذا الموقع ملائماً للغرض الذي أقيم من أجله .ومن الإعتبارات التي يجب مراعاتها في هذا الصدد مايأتي

١ -أن يكون البئر في جنوب المدينة أو القرية وإذا كان المورد سيتطلب إقامة عدد من الأبار فيجب تصميمها بحيث تكون ممتدة علي خط متجه من الشرق إلي الغرب لا من الجنوب إلي الشمال حتي لا تتأثر مياه البئر الشمالي من السحب من البئر الجنوبي .

٢ -ألا يكون البئر في أرض منخفضة حتي لا يكون عرضه لتجمع المياه السطحية حوله, كما في حالات الأمطار أو الفيضانات .

٣ -أن يكون بعيداً عن المساكن وفي وسط مساحة واسعة يمكن إقامة منشآت العملية بها ويكون من اليسر عمل حرم للمأخذ لمنع التلوث .

٤ -ألا يكون في منطقة الأرض بها غير سليمة أو بها شقوق .

٥ -أن يكفل ( وهو المهم ) إمداد المنطقة بكميات المياه اللازمة للسكان للشرب والأغراض المنزلية وكذا الأغراض التجارية والصناعية وأغراض البلدية .

## حماية البئر من التلوث :-

يجب عند اختيار مكان للبئر أن يكون بعيداً عن مصادر التلوث التي تأثر عليها أو التي يحتمل أن تؤثر عليها في المستقبل فيجب أن يكون البئر بعيداً عن مصادر التلوث بمسافة كافية وهذا البعد يختلف كما ذكرنا فيما سبق بإختلاف الظروف المحلية كما يتوقف علي عدة عوامل منها:-

١ -نوع وموقع مصدر التلوث

٢ -نوع البئر وتصميمه

٣ -مدرج المنسوب المائي

٤ -مسامية الأرض وتركيبها

٥ -مدي انخفاض مخروط المياه الجوفية أثناء سحب المياه من البئر

فمن ناحية نوع وموقع مصدر التلوث نجد أن التلوث الناتج من صرف متخلفات المراحيض يفوق في أثره التلوث الناتج من منزل قريب للبئر , كما أن نوع البئر وتصميمه له علاقة وثيقة بالتلوث . فالبئر الذي يسحب مياهه من المنطقة المشبعة ( كالأبار العميقة ) يكون أقل عرضة للتلوث من بئر يسحب مياهه من المنطقة الغير مشبعة .

ومدرج المنسوب المائي وحركة المياه الجوفية لهما دخل كبير في سير التلوث الأرضي , وكلما تفدينا المنطقة ذات المنسوب المتغير كلما كان ذلك أضمن و أفضل .

أما مسامية الأرض وتركيبها فتؤثر تأثيراً كبيراً علي مدي سير التلوث أو حجزه وقد سبق أن ذكرنا أنه كلما ذادت مسامية الأرض كلما أدي ذلك إلي سهولة تسرب المياه دون أن يتم ترشيحها الترشيح الكافي وبالتالي حجز ما بها من البكتريا .

أما عن مخروط السحب فلتفسيره نذكر أنه عند تشغيل الطلمبة لسحب المياه من البئر يترتب عليها انخفاض منسوب الماء في البئر ويؤدي ذلك إلي انخفاض مستوي المياه حول البئر وهذا ما يطلق عليه ( Cone of Depression ) أي مخروط السحب . ويترتب علي تكون هذا المخروط انحدار المياه نحو البئر من جميع الجهات مما يغير من الإتجاه الطبيعي للمياه الجوفية , ويسمح للتلوث السطحي الواقع في منطقة السحب بالتسرب بسرعة إلي البئر وكلما ذاد انحدار مخروط السحب نحو المأخذ كلما ذاد احتمال وصول المياه الملوثة إلي البئر . وكلما ذاد تشغيل الطلمبة الساحبة للمياه , كلما ذاد هذا الإنحدار – وعلي ذلك فإحتمال التلوث يزداد بازدياد السحب في حالة زيادة استهلاك المياه .

ويمكن القول عموماً أنه لحماية البئر من التلوث يجب أن تتوافر فيه الشروط الأتية:-

١ -أن يكون عمقه ٢٠ متر علي الأقل .

٢ -أن يكون بعده عن مصدر التلوث ٢٠ متراً علي الأقل وذلك بالطلمبات الميكانيكية . أما في الطلمبات اليدوية فيمكن أن يكون البعد ١٠ متراً من مصدر التلوث بشرط توفر العمق وبشرط أخذ عينات دورية للفحص البكتريولوجي والكيماوي فإذا ثبت تلوث المياه أوقف إستعمالها .

## المصادر االتى تؤدى الى تلوث الأبار

هناك عدة مصادر يمكن ان يترتب عليها تلوث . يمكن اعتبار المصادر الأتية كمصادر يترتب عليها تلوث الأبار اذا لم تستوف الابعاد والمسافات الواجبة

خزانات المراحيض الغير صماء – بيارات الصرف – أكوام السباخ –حظائر الماشية –المصارف – الجبانات – البرك والمستنقعات – المياه الراكدة حول المأخذ أو بالقرب منة – منازل الفلاحين – مزارع المجارى – الاراضى الزراعية التى تسمد بالسماد العضوى – العيوب التى قد توجد فى التصميم واقامة الآبار ( شروخ فى المواسير ، قدمها ، تأكلها ) ترنشات الصرف .

ويجب على الذين يعملون فى الميدان الصحى ان يراعوا عند اختبار مورد المياه , أن يتأكدوا من عدم وجود أحد هذه المصادر أو غيرها على مسافة ٢٠ مترا فى حالة العمليات الميكانيكية أو ١٠ متر فى حالة الطلمبات اليدوية . وأن يذكر فى أورنيك ارسال العينة جميع البيانات عن مصدر التلوث واحتمال تأثيرة على مياه البئر سواء وقت أخذ العينة أو فى المستقبل .

## الآبـــار

الآبار هى الوسيلة الصناعية التى نحصل بها على المياه الجوفية , وقد عرفت الآبار منذ قديم الازل الا أنها لم تكن متوافرة الشروط الصحية سواء فى أقامتها أو صيانتها . وكانت غالبا من النوع المكشوف الذى تستخرج منة المياه بواسطة الدلو , وهذا النوع من الآبار المكشوفة مازال موجودا فى بعض القرى الا أن عددا كبيرا منها قد تم ردمه وحلت محله مشروعات المياه الكبرى . وان كان هناك بعض القرى المحرومة من المياه الصالحة للشرب مازال بها عدد من هذة الآبار .

## الاحتياطات الواجب اتخاذها اثناء دق الأبار أو اقامتها

يجب اتخاذ كافة الاحتياطات التي تكفل عدم تلوث المياه أثناء دق أو اصلاحة . وفيما يلي نورد هذه الاحتياطات :

١ يجب أن يحرص الأشخاص والعمال القائمين بدق البئر علي عدم تلويث مجاوراته لمسافة ١٠٠ متر حوله . كما يجب عدم استخدام مواسير أو أدوات تكون قد تلوثت بالشحم أو السماد. وأن يتخذوا جميع الاحتياطات الكافيله بدرء التلوث الخارجي عن البئر .

٢ يجب غسل جميع المواسير والقاسون المستعمل بمحلول مركز من هيبوكلوريت الصوديوم ويطلق عليه ( جير كلور) قبل انزالها في الأرض .

٣ إذا استعمل الزلط في إقامة البئر فيجب غسله في محلول مركز من هيبو كلوريت الصوديوم ( جير الكلور ). والنسبة المستعمله في هذه الحالة هي نصف كيلو من هيبو كلوريت الصوديوم إلي ٦ لتر من الماء

٤ بعد الانتهاء من دق البئر وإنزال المواسير المجلفنه يجب تعقيم المياه قبل أخذ العينات وذلك بإستعمال مادة هيبو كلوريت الصوديوم قوة ٢٥% وستذكر فيما يلي طريقة تعقيم البئر .

## طريقة تعقيم الأبار

١. يمزج ٥٧ جم من مسحوق جير الكلور قوة ٢٥% في٠ ٢ لتر ماء لكل ٤٠٠ لتر من المياه داخل الماسورة , ويتم تحضير هذه المادة بأن تضاف كمية بسيطة من الماء إلي كمية جير الكلور ويقلب بهدوء حتي نحصل علي عجينة , ثم نضيف إليها كمية الماء ( ٢٠لتر لكل ٤٠٠ لتر من الماء في الماسورة ) ويقلب السائل بهدوء لمدة ١٠ إلي ١٥ دقيقة ويترك ليرسب ويأخذ السائل الذي يحتوي علي الكلور الفعال لإستعمالة في التعقيم .

٢. يضاف السائل الناتج من رقم (١) إلي المياه الموجودة في الماسورة ويترك لمدة ٤٨ ساعة وفي خلال هذه الفترة يبدء في إدارة الطلمبه حركة بسيطة تتلوها حركة عكسية دون إخراج الماء وهذه العملية تتيح مزج المطهر بالماء كما يتيح لهذه المياه تعقيم جدران الماسورة الداخلية أثناء صعودها عند إدارة الطلمبه أو نزولها عند إيقاف الإدارة بحركة عكسية ويجب إجراء هذه العملية عدة مرات .

٣. يبدأ بعد مضي الـ ٤٨ ساعة المتقدمة في إدارة الطلمبة إدارة تامة لتفريغ محتويات الماسورة حتي يزول كل أثر الكلور في الماء ( تكون فترة الإدارة ٢٤ ساعة )

٤. إذا تتطلب الأمر أخذ عينات فتكون علي الأساس الأتي :-

- العينة الأولي بعد ٢٤ ساعة إدارة مستمرة من انتهاء فترة التعقيم وتفريغ الماسورة
- العينة الثانية بعد إدارة ٢٤ ساعة مستمرة من العينة الأولي .

- العينة الثالثة بعد إدارة الطلمبة ٢٤ ساعة بعد العينة الثانية .

## حماية موارد المياه الجوفية

من أهم الأعباء التي تقع علي عاتق القائمين بالرقابة علي موارد المياه وصيانتها العمل علي اتخاذ الاحتياطات الكافيله بحماية المورد من التلوث وهذه الاحتياطات متنوعة يمكن أن نلخص بنودها الأساسية في الأتي :-

١ يجب رفع القاسون وانزال المواسير يجب عمل تغطية من الأسمنت أو أي مادة صماء حول الماسورة بحيث تشمل هذه التغطية الجزء غير المخرم منها ويتم عمل هذه التغطية بالأسمنت بطريقة الضغط وتشترط بعض الاشتراطات في الأسمنت المستعمل وفي طريقة تحضيرة بحيث يكون طبقة متماسكة حول الماسورة ولا يكون به شقوق تسمح للمياه السطحية بالتسرب إلي المأخذ .

٢ يستحسن عدم وضع الماكينة فوق ماسورة الغز مباشرةً بل يتم تركيب الماكينة علي مداد من ماسورة الغز وذلك لإمكان إصلاح أي عيب يطرأ علي الماسورة دون حاجة إلي رفع الماكينة .

٣ يجب عمل دكة أسمنت حول ماسورة المأخذ لمسافة دائرة نصف قطرها ٥ أمتار من كل ناحية وتكون هذه الدكه مائله من البئر إلي خارجه بشكل هرمي وذلك لحماية المأخذ من تسرب المياه السطحية حوله .

٤ يجب عمل حرم للبئر لمسافة ٢٠ متر من جميع الجهات وأن يتم عمل سور متين علي هذه المسافة مع إزالة جميع مصادر التلوث التي تكون موجودة في هذا الحرم

٥ يجب عدم وضع مواسير شبكة توزيع المياه أو أي محابس بجانب مواسير المجاري , بل تكون شبكة مواسير المياه مستقله عنها . مع مراعاة نظافة هذه المواسير قبل وبعد وضعها .

٦ يجب عمل طريقة صرف صحية للمياه المستعمله في تبريد الماكينات أو المياه الفائضة من الصهريج أو المتخلفة من غسيل الصهريج بحيث تصرف هذه المياه بعيداً عن المأخذ .

٧ -إذا وجدت مراحيض لموظيفي العمليه أو دورات المياه فيجب تصميم طريقة صرف صحية لها مع مراعاة أن تكون بيارات الصرف أو الترنشات والخزانات علي مسافة لا تقل عن ٢٠ متر من أبار العملية .

## خزانات وصهاريج المياه

إن إقامة عمليات المياه العامة يتطلب عمل خزان للمياه وتتوقف ساعة الخزان وحجمة علي نوع العملية وقوتها والإستهلاك اليومي وإعتبارات فنية أخري وتتم إقامة هذا الخزان بحيث يكون أعلي من منزل في المدينة حتي تصل المياه إلي الأدوار العليا من المنازل . ولما كانت المياه قد تبقي في هذا الصهاريج مدة طويلة وبإستمرار الإستعمال يتعرض الخزان للتلوث ويؤدي ذلك بالتالي الي تلوث المياه المستعمله . ولذا وجب علي القائمين علي عمليات المياه والمشرفين مولاة التأكد من حالة هذه الخزانات وعدم تلوثها . ومن الأمور الهامه التي يجب مراعاتها في هذه الخزانات ما يلي :-

١ -يجب أن يكون موقع الخزان في مكان نظيف وان يكون الوصول اليه سهلاً

٢ -يجب أن يغطي بغطاء محكم لمنع وصول الأتربه إليه .

٣ -أن تعمل له ماسورة للمياه الفائضة بحجم مناسب حتي يمكن إبطار تشغيل الماكينة إذا ما إمتلأ الخزان .

٤ -يجب أن يصمم الخزان بحيث تسحب المياه من أعلي القاع بقليل ليمكن للرواسب أن تتجمع في هذه المسافة . ويكون من السهل إزالتها عند غسله وتعقيمه .

٥ -إذا كان الخزان من الصاج فيجب أن يتم دهانه ببويه مانعة من الصدأ وأن يتم تحليل هذه البويه بمعامل وزارة الصحة قبل إستعمالها للتأكد من خلوها من المواد السامة وإلي جانب هذه الإستراطات العامة فيجب ضرورة تعقيم الخزان بمادة جير الكلور مرتين في العام ما لم يطرأ طارئ يستدعي التعقيم ( كما في حالة الأوبئه ) وفيما يلي طريقة تعقيم الخزانات :-

## طريقة تعقيم الخزانات

١. يجب إعداد جميع مواد النظافة والتعقيم ( جير الكلور ) قبل البدء في العملية
٢. يفرغ الخزان من المياه مع الإبقاء علي كمية منها بإرتفاع ٤٠: ٥٠ سم تستعمل في تنظيف الخزان , أرضيته وجدرانه .
٣. العمال الذين يقومون بعملية التنظيف يجب أن يلبسوا أحذيه من المطاط ذات الرقبه الطويله بعد وضعها لبضع دقائق في محلول مخفف من كلورور الجير ( واحد في الألف تقريباً ) أي بنسبة ملعقه كبيرة في جردل ماء سعته حول ١٠لتر .
٤. بعد إزالة جميع المواد الملتصقة بأرضية وجدران الخزان بالفرش , تطرد مياه الغسيل بالعادم ثم تمرر كمية مياه جديده وتطرد , وتكرر هذه العملية حتي لا تبقي أية رواسب بأرضية الخزان.
٥. يعقم الخزان بإستعمال مسحوق جير الكلور الذي قوته ٢٥% ( علي الأقل ) وذلك باستحلابه بمعدل ١٠ جرام لكل متر مكعب حسب سعة الخزن فمثلاً لتعقيم خزان سعته ١٠٠ متر يستحلب كيلو جرام في جردل علي دفع ويضاف إلي مياهه أثناء ملئه ويترك ساعتين , ثم يفرغ ويبدء في ملئه للإستعمال .)
٦. في غير فترات غسيل وتعقيم الخزان يمكن عمل طرد من آن لأخر ( Periodic flushing) للتخلص من أية رواسب موجودة بأرضية الخزان وذلك بإدخال كميات قليله من المياه وهو فارغ ثم طردها للعادم دون وصول أي شخص بداخله وفي هذه الحالة لا داعي لتعقيمه .

## مزايا المياه الجوفيه

للمياه الجوفيه يمكن أن نلخصها في الأتي : –

١ -أن الأمر لا يتطلب فيها إقامة منشأت بنايه كبيرة للمياه لأن الأرض تعتبر مخزن للمياه .

٢ -أنها أقل عرضه للتلوث خاصةً تلك التي نحصل عليها بواسطة الأبار العميقه لأن قدرة الأرض علي ترشيح المياه تكون كافية لصلاحية المياه المسحوبه من البئر إلا في بعض الحالات الأتية:–

أ –إذا كانت طبيعة الأرض مشبعه بالمواد العضوية

ب إذا كانت هناك خزانات لصرف مجاري أو أي مصادر تلوث أخري قريبة من البئر

ج- إذا كان هناك شقوق في سطح الأرض تتسرب منها المياه السطحية إلي المأخذ

٣- لا يتطلب الأمر إجراء عملية ترسيب أو ترشيح للمياه الجوفية ولاكن قد يحتاج الأمر إلي تعقيمها بالكلور كاإجراء وقائي .

٤-إن أقامة بئر أو عدة أبار للمياه يحتاج إلي مساحة قليله من الأرض كما أن النفقات أقل . ولذلك فهي مفضله في حالة مشروعات المياه التي تهدف إلي تغذية أكبر عدد ممكن بأقل النفقات .

## مساوئ المياه الجوفيه

١ -إحتمال عدم الحصول علي الكميات الكافية من المياه من الأبار الخاصة في المدن الكبيرة التي يزداد فيها الاستهلاك منما يتطلب ضرورة تدبير موارد أخري .

٢ -أن المياه غالباً غسره ولابد من عمل تيسير لها إذا ذاد هذا العسر كما أن طعمها قد يكون ردئ . وإذا كانت المياه الجوفيه ستسعمل في أغراض الصناعه فإن العسر المؤقت يؤدي إلي أملاح كربونات وكبريتات الكالسيوم وهي تؤدي إلي خفض درجة الحرارة في الغلايات وقد يتسبب عنها إنفجار هذه الغليات .

٣ -وجود الحديد والمنجنيز بنسبة كبيرة يتطلب ضرورة إزالته حتي لا يكون للماء شكل أو طعم غير مرغوب فيه وذلك علاوه علي تأثير الحديد في الأقمشه .

# الفصل الثالث

## المعالجة التقليدية لمياه الشرب

# الباب الثالث

# المعالجة التقليدية لمياه الشرب

## الهدف من عمليات التنقية

يقصد بالتنقية التخلص من كل او بعض المواد الغريبة سواء كانت ذائبة او عالقة او غروية . حيث ان المياه السطحية معرضة لعوامل كثيرة تؤدى الى تلوثها فتصبح غير صالحة للاستعمال الا بعد تنقيتها

## عمليات المعالجة

تمر المياه الخام بعدة عمليات لمعالجتها لتكون صالحة للشرب وهى:

١. المعالجة اولية وتنقسم إلى:

- التصفية
- التطهير المبدئى
- المعالجة الكيماوية المبدئية
- الترسيب المبدئى (اختيارى)

٢. المعالجة الرئيسية وهى تلى المعالجة الاولية وتنقسم إلى:

- الترويب
- التنديف
- الترسيب
- الترشيح
- التطهير/ التعقيم
- التخزين
- الضخ

| العمليـــــــة | الهدف من إجراء العملية |
|---|---|
| مياه عكرة | |
| مصافى | إزالة الأوراق والأغصان والأسماك وغير ذلك |
| الكلور الميدانى | التخلص من معظم الكائنات الحية المسببة للأمراض والتحكم فى الطعم والرائحة |
| الكيماويات | تجميع الجسيمات الدقيقة جداً لتكوين جسيمات أكبر |
| المزج السريع | خلط الكيماويات مع المياه العكرة التى تحتوى على الجسيمات الدقيقة التى لم ترسب أو ترشح |
| الترويب والتنديف | تجميع الجسيمات الدقيقة والخفيفة معاً لتشكيل جسيمات أكبر تساعد فى عمليات الترويق والترشيح |
| الترسيب | ترسيب الجسيمات الأكبر العالقة |
| الترشيح | ترشيح الجسيمات العالقة المتبقية |
| الكلور النهائى | التخلص من الكائنات العالقة المسببة للأمراض وتوفير الكلور المتبقى اللازم لشبكات التوزيع |
| خزان المياه | توفير وقت تلامس الكلور بغرض التطهير وتخزين المياه لمواجهة الطلب المتزايد |
| المياه المرشحة | |

# الترسيب

الغرض من عملية الترسيب هو العمل على ترسيب اكبر نسبة ممكنة من المواد العالقة و التى لها ثقل اكبر من دفع الماء و ذلك عن طريق تركها ترسب تحت تاثير وزنها . و عملية الترسيب اما ان تكون طبيعية , اى تحت تاثير وزن المواد العالقة بدون اضافة اى مواد و تسمى هذه العملية بالترسيب الطبيعى. او تكون باضافة مواد مساعدة كيماوية للماء لتساعد على تجميع المواد الغير قابلة للترسيب و تسمى هذه العملية بالترسيب بمساعدة المروبات

** العوامل المؤثرة على كفاءة عمليات الترسيب منها:-

١. تركيز المواد العالقة
٢. شكل و حجم و كثافة المواد العالقة
٣. درجة حرارة و لزوجة المياه
٤. مدة بقاء الماء فى الاحوض

## نظرية الترسيب الطبيعى

يحدث الترسيب الطبيعى نتيجة هبوط الحبيبات منفردة, و يحدث التؤسيب تحت تاثير وزن الحبيبة فقط و يحدث مقاوة اثناء هبوط الحبيبات المنفردة نتيجة لزوجة المياه و حيث ان قانون نيوتن يوضح انه اذا توازنت القوى المؤثرة على الحبيبات .لا يحدث عجلة تسارع و لكن تحدث سرعة هبوط ثابتة

و تعتمد نظرية الترسيب على تخفيض سرعة المياه الداخلة الى احواض الترسيب الى ادنى درجة بحيث يتم ايقاف الحركة الافقية للحبيبات العالقة و العمل فقط بالحركة الافقية مما يساعد على سرعة الترسيب قبل خروج المياه من احواض الترسيب و دخولها الى المرشحات و التى تقوم بازالة المتبقى من الحبيبات التنى لم يتم اذالتها اثناء عملية الترسيب .

و تقاوم الحبيبات الغير قابلة للترسيب عملية الترسيب لسببين اساسيين هما :-

١. حجم الحبيبات نفسها
٢. القوى الطبيعية بين الجسيمات

## أ . حجم الجسيمات

تنقسم الجسيمات من حيث الحجم الى

١. مواد معلقة

٢. مواد غروية

٣. مواد ذائبة

قابلية ترسيب الجسيمات في المياه

| قطر الحبيبات (مم) | النوع | زمن الترسيب* | القابلية للترسيب |
|---|---|---|---|
| ١٠ | حصى | ١ ثانية | قابلة |
| ١ | رمل | ١٠ ثوان | قابلة |
| ٠,١ | رمل ناعم | ٢ دقيقة | قابلة |
| ٠,٠١ | طمى / طحالب | ٢ ساعة | غير قابلة |
| ٠,٠٠١ | بكتريا | ٨ أيام | غير قابلة |
| ٠,٠٠٠٢ | مواد غروية | سنتين | غير قابلة |
| ٠,٠٠٠٠٢ | فيروسات | ٢٠ سنة | غير قابلة |

.

** زمن الترسيب هو المدة التقريبية للرسوب لمسافة ١ م فى المياه تحت تاثير الجاذبية فقط

## أولا : المواد الصلبة المعلقة:

هى جسيمات ينقلها معه الفعل الطبيعى للمياه الجارية وهى أكبر من ( ٠.٠٠١ مم) والجسيمات التى لا تتجاوز( ٠.٠١ مم ) حجماً لا تترسب بسرعة وهى تسمى فى عمليات معالجة المياه "مواد غير قابلة للترسيب" أما الأكبر حجماً (أكبر من >٠.٠١ مم) فتوصف بانها قابلة للترسيب , حيث أنها تترسب تلقائيا إلى القاع أو حوض الترسيب فى غضون أربع ساعات

## ثانيا: المواد الصلبة الغروية:

هى جسيمات الطمى الدقيق والبكتريا والجسيمات المسببة للون والفيروسات وهذه الغرويات لا تترسب تلقائيا في مدة زمنية معقولة كما هو موضح بالجدول وبالرغم من أنه لا يمكن رؤيتها بالعين المجردة إلا ان تاثيرها قد يشاهد عند مرورها خلال عمليات المعالجة إذا لم تتروب تندف كما يجب

## ثالثأ : المواد الصلبة المذابة:

هى أى مواد عضوية أو غير عضوية "الاملاح والكيماويات التى من أصل نباتى أو حيوانى أو غازات) يمكن ذوبانها فى المياه, والمادة الصلبة المذابة هى جسيمات صغيرة جداً (فى حجم الجزئ) ولا يمكن رؤيتها بالعين المجردة وغالبا تكون المعادن المذابة فى الماء مواد صلبة غير قابلة للترسب، وتسبب مشكلات للصحة العامة ومشكلات أخرى مثل الطعم أو اللون أو الرائحة وما لم تحول إلى مواد مترسبة بواسطة وسيلة كيميائية أو فزيائية فلا يمكن إزالتها من الماء

ومن الاسباب الرئيسية التى تجعل الجسيمات غير قابلة للترسيب هو صغر حجمها والذى يمكن توضيحة بالمثال التالى:-

## مثال:

من المتوقع أن يترسب حجم من رمل خشن على شكل مكعب طول ضلعه ١ مم بحيث يهبط نحو ٠.٣ متر كل ٣ ثوان والآن لنفرض أن حبيبة الرمل قد طحنت إلى عدة جسيمات أصغر, كل منها مكعب طول ضلعه ٠.٠٠٠٠١ مم (هذا الطحن يماثل قوى التعرية الطبيعية) فإن الوزن الاجمالي لكل الجسيمات المطحونة يساوى وزن الحبيبة الاصلية للرمل الخشن.

وبمقارنة المساحة السطحية فى الحالتين نجد انها قد زادت من ١ مم $^{٢}$ وهى مساحة رأس دبوس كبير إلى ٦ م$^{٢}$ وهذه الزيادة فى المساحة السطحية تسبب زيادة هائلة فى قوى المقاومة والتى تقاوم الترسيب الطبيعى التلقائى وطبقاً للجدول فبدلاً من ان ترسب تلك الجسيمات الدقيقة إلى عمق٣ . ٠ متر فى ٣ ثوان فإنها تستغرق الآن نحو ٠ ٢ سنه لترسب إلى نفس العمق لذلك فمن اللازم لتعجيل الترسيب أن يتم تجميع الجسيمات الدقيقة معاً لتكون جسيمات أكبر يمكن ترسيبها بسهولة وسرعة.

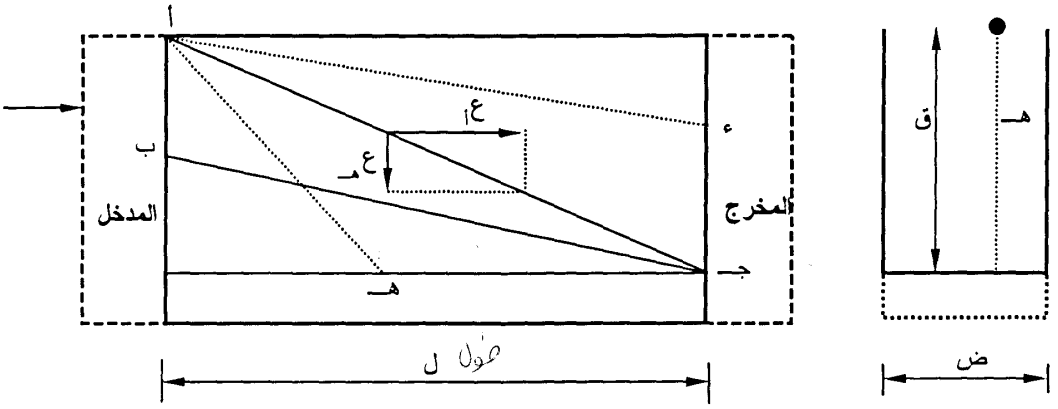
## ب . القوى الطبيعة

تحمل عادة الجسيمات الموجودة فى المياه شحنات كهربية سالبة وكذلك فإنه توجد هناك قوة تنافر بين أى جسمين متماثلين فى الشحنة. وفى عملية معالجة المياه تسمى قوة التنافر الكهربية جهد زيتا Zp وهى قادرة على إبقاء الجسيمات الغروية الصغيرة جدا متباعدة بعضها عن بعض ومعلقة فى الماء. وتوجد قوة أخرى هى قوة فان درفال بين جميع الجسيمات فى

الطبيعة وهى تعمل على جذب أى جسمين معا. وقوة الجذب هذه تعمل فى اتجاه مضاد لجهد زيتا وطالما كان جهد زيتا أقوى من قوة فان درفال فإن هذه الجسيمات تظل معلقه.

ومن الناحية النظرية البحتة فإنه يمكن توضيح عملية الترسيب كما هو موضح فى الشكل . كمدخل لتفهم أسس تصميم عملية معالجة المياه. ذلك على أساس أن المواد العالقة متجانسة التوزيع فى الماء. وبفرض أن سرعة المياه بما فيها من مواد عالقة فى الاتجاه الأفقى (ع) وأن تصرف المياه يساوى( ك) وأن عرض الحوض (ض) وعمقه ( ق) وتكون مساحة مقطعه (ضXق) وتكون سرعة المياه الافقية = معدل التدفق/ مساحة المقطع.

$$ع = \frac{ك}{ض X ق}$$




وتكون هى أيضأ السرعة الافقية للمواد العالقة بمختلف أحجامها

ع أ سرعة الأجسام العالقة فى الاتجاه الأفقى

ع هـ سرعة الأجسام العالقة فى الاتجاه الراسى (سرعة الهبوط)

ق عمق الحوض

ض عرض الحوض

ح المساحة السطحية لحوض الترسيب

أء مسار المواد التى لا ترسب وتخرج من الحوض

ب جـ مسار المواد التى لا يرسب منها إلا ما كانت نقطة دخوله بالقرب من القاع

أ هـ مسار الأجسام ذات الكثافة والحجم الأكبر والتى ترسب بسرعة

أما سرعة المواد العالقة فى الاتجاه الرأسى ع أ فتكون مختلفة حسب حجم الجزيئات وكثافتها.
ومن الشكل نستنتج أن:

$$\frac{ع_أ}{ع_هـ} = \frac{ل}{ق}$$

$$\therefore ع_هـ = \frac{ق}{ل} \times ع_أ$$

وحيث أن

$$ع_أ = \frac{ك}{ض \times ق}$$

$$\therefore ع_هـ = \frac{ك}{ض \times ق} \times \frac{ق}{ل} = \frac{ك}{ض \times ل}$$

أى أن:

$$\text{سرعة الهبوط الرأسية للحبيبات} = \frac{\text{معدل التدفق (التصرف)}}{\text{المساحة السطحية لحوض الترسيب}}$$

ويتضح من ذلك أن المواد العالقة التى لها سرعة هبوط رأسية مساوية أو أكثر من ع هـ يتم ترسيبها فى الحوض. وعلى العكس لا ترسب المواد إذا كانت سرعتها الراسية أقل من ع هـ.
ومن ذلك نرى أن المساحة السطحية لحوض الترسيب(ض X ل) لها تاثيرمباشر على كفاءة الترسيب. فكلما ذادت المساحة السطحية قلت ع هـ وزادت كفاءة الحوض معدل التدفق.و تسمى ع هـ بمعدك التحيل اسطحى لحوض الترسيب. وتتخذ كأساس من أمس التصميم لهذه الاحواض. وتكون وحدتها متر/ساعة

ومن الشكل يتبين أن المسار أء هو مسار المواد التى لها سرعة هبوط رأسية أقل من ع هـ وهذه لا ترسب فى الحوض وتخرج منه , ولا يرسب منها إلا ما يدخل منها قرب قاع الحوض والذى يمثله المسار (ب جـ). أما المسار أ هـ فيبين مسار المواد التى لها كثافة وحجم أكبر، وترسب بسرعة لأن سرعتها الرأسية تكون أكبر من ع هـ

## الترسيب بمساعدة المرويات

عندما تبين عدم ملاءمة الترسيب الطبيعى للطلبات المتزايدة على المياه ء وأيضا لعدم جدواه فى ترسيب الحبيبات الدقيقة والخفيفة و خاصة المواد الكلويدية والعوالق الطينية والكائنات الحية الدقيقة والمواد العضوية الناتجة من تحلل النباتات المائية ء وذلك إما لصغر سرعة الترسيب بشكل كبير وبالتالى تحتاج إلى مدة مكث كبيرة .و لأن الجسيمات الموجودة بالماء تحمل شحنة كهربية سالبة وبالتالى يحدث تنافر بينها لتماثل الشحنات فتبقى الجسيمات متباعدة عن بعضها. لذلك تضاف إلى المياه مواد كيماوية (مروبات) تعمل على تجميع هذه الحبيبات الرفيعة , كما سبق توضيحه . وبعد عمليتى المزج السريع والمزج البطئ تمر المياه فى أحواض الترويق حيث ترسب الندف المتكونة فى احواض الترويق بما جذبت إلى سطحها من مواد عالقة إلى قاع الحوض.

ولا تختلف أحواض الترويق فى تصميمها عن أحواض الترسيب الطبيعى، كما أن العوامل المؤثرة على كفاءة الترسيب فيها لاتختلف عن احواض الترسيب الطبيعى ولكن فى وقت أقل جداً , وبالتالى تكون أحجام هذه الآحواض أصغر كثيراً. وبالرغم من اختلاف أنواع وطرازات احواض الترويق إلا أنها تتفق فى الاسس الرئيسية للتصميم وان اخلفت فى بعض التفاصيل .

## العوامل المؤثرة فى عملية الترسيب

تتاثر عملية الترسيب بعدة عوامل أهمها:-

١. حجم الحبيبات: وطريقة توزيعها، فكلما زاد حجمها ووزنها, ازدادت كفاءة الترسيب .
٢. شكل الحبيبات :- فكلما اقترب شكلها من الكروى كلما كان ترسيبها اسرع وأكفء
٣. كثافة الحبيبات: فكلما زادت كثافتها زادت كتلتها بالنسبة لحجمها وزادت سرعة رسوبها وبالتالى كفاءة الترسيب .
٤. درجة حرارة الماء: فكلما ارتفعت درجة حرارته قلت كثافته ولزوحته وبالتالى زادت سرعة رسوب الحبيبات وزادت كفاءة الترسيب .
٥. الشحنة الكهربائية للجسيمات: والتى تكون دائما سالبة الشحنة، وعند معالجة المياه بالشبة موجبة الشحنة , يحدث تجاذب بين الجسيمات السا لبة والموجبة مما يساعد على ترسيبها وزيادة كفاءة الترسيب .
٦. سرعة سريان الماء فى الحوض: فكلما قلت سرعة الماء , زادت كفاءة الترسيب، ويفضل ألا تتجاوز السرعة الافقية فى الحوض( ٣٠ سم/ دقيقة)
٧. مدة بقاء الماء فى الحوض : مدة المكث فكلما ذادت المدة ذ ادت جودة الترسيب. ومن النواحى الاقتصادية والعملية أن تكون مدة المكث فى حدود ٣ : ٤ ساعات و حيث أن زيادة المدة أكثر من اللازم لا يزيد من كفاءة الترسيب إلا بنسبة بسيطة.

$$\text{مدة المكث} = \frac{\text{حجم حوض الترسيب}}{\text{معدل التصرف خلال الحوض}}$$

النسبة بين طول وعرض حوض الترسيب فى الأحواض المستطيلة. وذلك لاقلال فرص تكون مناطق راكدة أو ميتة عند زيادة عرض الحوض .

# الفصل الرابع

## عملية الترويب و التنديف

# الباب الرابع

# عملية الترويب والتنديف

تتكون الشوائب الموجودة فى المياه نتيجة التآكل الأرضى الناتج من احتكاك المياه مع قطاع المجرى المائى، وتتكون الشوائب من دقائق صغيرة كما تلتقط المياه مع الاحتكاك ايضا كميات من الاملاح وبقايا النباتات المتحلله كما توجد شوائب أخرى إضافية ناتجة من التلوث الهوائى ومخلفات المصانع و فضلات الحيوانات. ولذلك فالمياه السطحية تتلوث بتاثير الطبيعة و الانسان. وتحتوى المياه على مواد عالقة ومواد ذائبة عضوية(من اصل نباتى اوحيوانى) وكذلك على المواد غير العضوية وكافة الاشكال البيولوجية( كالبكتريا والنباتات المائية والهائمات ) وتسمى هذه المواد" المواد العالقة الصلبة" وتكون قدرأ كبيرأ من مكونات الشوائب. أما الاجزاء الكبيرة الحجم مثل الرمل والطمى الثقيل فيمكن ازالتها من المياه بتركها للترسيب بواسطة الجاذبية الارضية و تسمى هذه المواد بالمواد الصلبة القابلة للترسيب

و يحدث ترسيب للمواد الصلبة طبيعيا عند تخزين المياه فترة كافية فى الخزانات او فى الانهار و البحيرات ذات السرعة البطيئة . لكن الاجزاء الصغيرة جداً مثل المواد الغروية والطمى الدقيق والبكتريا التى لا تترسب بسهولة وتحتاج إلى معالجة لتجعل جزيئاته أكبر وذات قابلية للترسيب. وتسمى هذه المواد بالمواد غير القابلة للترسيب أو المواد الغروية .

## الحاجة الى الترويب و التنديف

الغرض من تلك العمليتين هو ازالة المواد الشائبة و خاصة غير القابلة للترسيب الطبيعى و ازالة العكارة من المياه المراد معالجتها

وتعمل المواد الكيماوية (المروبات) على إزالة المواد الغير قابلة للترسيب بعملية الترويب ونلك بأن تجعل هذه الجزئيات تتجمع مع بعضها البعض لتكون الندف الصغيرة ثم تتجمع الندف الصغيرة مع بعضها البعض لتكوين نفدف كبيرة و ثقيلة قادرة على الترسيب. تخلط المواد الكيماوية بالماء العكر الذى يحتوى على جسيمات دقيقة لا تترسب و لا تترشح بسهولة و تعمل على جعل الجسيمات الدقيقة الشديدة الصغر تتجمع وتتكتل معأ فى صورة أجسام أكبر (ندف) للمساعدة فى إتمام عمليتى الترسيب والترشيح ثم ياتى الترسيب بعد ذلك ليرسب الجسيمات العالقة الكبيرة ثم الترشيح ليحتجز ويستخرج الجسيمات العالقة الباقية.

## و صف عملية الترويب و التنديف

تؤدى عملية التر ويب/ التنديف إلى تعادل أو تقليل جهد زيتا للمواد الصلبة غير القابلة للترسيب بحيث يمكن لقوة فان درفال أن تبدأ فى جذب الجسيمات معأ وعندئذ تتمكن الجسيمات الغير قابلة للترسب من التجمع فى مجموعات صغيرة من الندف الدقيقة . ورغم أن هذه الجسيمات أكبر من الغروية الأصلية إلا أنها ضعيفة التماسك مع بعضما وتكون الندف الدقيقة الغير مرئية للعين المجردة وتظل غير قابلة للترسيب ويودى فعل التقليب البطىء الذى يحدثه التنديف إلى تجميع الجسيمات الندفية معأ لتكوين جسيمات ندفية كبيرة وثقيلة نسبيأ يمكن ترسيبها .و ترشيحها بسهولة، والجسيمات الندقية الهلامية القوام تكون مرئية فى العادة وتبدو مثل ندف القطن او الصوف.

## استخدام المرويات(مواد الترويب)

وحيث أن معظم الجسيمات غير المرغوبة والمطلوب إزالتها من المياه سالبة الشحنة فبالتالى يجب أن تكون المرويات المستعملة فى معالجة المياه ذات شحنه موجبة , فالشحنات المرحبة تعادل الشحنات السا لبة وتعجل التر ويب .

وتحتوى بعض المروبات على أيونات ذات شحنات موجبة أكثر من البعض الأخر فنجد أن تلك المشتملة على أيونات ثلاثية التكافوء مثل الالمونيوم و الحديد ($Fe^{+++}$, $AL^{+++}$) أشد فعالية بمقدار يتراوح من ٧٠٠ – ١٠٠٠ مرة (كمروبات) من الايونات الأحادية التكافو مكر الصوديوم ($Na^{+}$) وأشد فعالية بمقدار ٥٠- ٦٠ مرة من الأيونات ثانية التكافو ط الكالسيوم $Ca^{++}$ ومن المواد الشائعة المستخدمة فى عملية التر ويب هى كبريتات الالمونيوم و اسمها الدارج الشبة $AL_2(SO_4)_3.nH_2O$, و عند ذوبان هذا المروب و تاينه فى الماء فإنه يكون الأيونات ثلاثية التكافو للألومونيوم $Al^{+++}$ وتتراوح نطاق الجرعة من ( ٥ – 100) مللجم/ لتر .

# أولا: الترويب

## تعريف عملية الترويب

توضح كلمة الترويب التأثير الناتج عندما تضاف مواد كيماوية معينة للمياه الخام المحتوية على مواد بطينة الترسيب ومواد لا تترسب طبيعياً , ويمكن تعرف التر ويب على أنه بداية استجابة الجسيمات بطينة الترسيب فى المياه

الخام لعمل ندف دقيقة عند إضافة المروب للمياه مشكلاً سحابة هلامية ويسلزم ذلك عملية خلط سريع لضمان توزيع متماثل للمادة المروبة .

## الخلط السريع

هو مزج المواد المروبة مع المياه الخام والغرض من المزج السريع هو التوزيع المتماثل للمواد الكيماوية المروبة خلال المياه وتحدث هذه العملية فى وقت قصير جداً عدة ثوان لم وتكون النتيجة الأولية لهذه العملية هى تكوين

جزيئات صغيرة جدا. فالتلامس الأول للمروب مع المياه هى الفترة الزمنية الأكثر حرجاً فى عملية التر ويب بأكملها وذلك لأن تفاعل الترويب يحدث بسرعة، وعليه فمن المهم أن يتلامس المروب والجسيمات الغروية فوراً وبعد إضافة المر وب.

## وصف العملية الترويبية من الناحية الكيماوية:

تعتبر العملية الترويبية من العمليات المعقدة وهى عملية فيزيائية وكيميانية تحث بين قلوية المياه (وهى قدرة المياه على معادلة الأحماض) الناتجة من وجود مكونات مثل الكربونات والبيكربونات والهدروكسيدات واليورات والسيليكات والفوسفات بالمياه وبين المادة المروبة.

ويساعد تقليل الرقم الهير وحينى للمياه( pH( ٨ على تهيئة الوسط للشحنة الموجبة وهو المناسب للتفاعل مع الجزيئات و الغرويات السا لبة الشحنة مكوناً ندف غير ذانبة التى تعمل على إزالة الشوائب من المياه.

ويعتبر مدى الرقم الهير وحينى من ( ٥.٥ – ٧.٥ ) مدى مناسب لعملية الترويب وتعطى الشبة أفضل نتائج فى هذا المدى حيث أن المادة المروبة تتفاعل مع جزء من القلوية فى الماء، والقلوية المتبقية بعد التفاعل والموجودة فى المياه تعمل كمنظم تمنع الرقم الهيدروحينى للمياه من التغير وبالتالى تساعد على الترسيب الكامل للمادة المروبة ويمكن تمثيل معادلة التفاعل

كالآتى:

**كبريتات الألومونيوم (الشبة) + القلوية (بيكربونات الكالسيوم) ←——— هيدروكسيد الألومونيوم (الندف) + كبريتات الكالسيوم + ثانى أكسيد الكربون**

وفى حالة عدم وجود القلوية فى المياه يضاف إليها الصودا الكاوية أو الجير المطفأ Ca(OH)2وتكون المعادلة كالتالى :

**كبريتات الألومونيوم + الجير ( $Ca(OH)_2$) ←—— هيدروكسيد الألومونيوم (الندف) + كبريتات الكالسيوم**

وبصفة عامة فإن المشغل لا يستطيع التحكم فى القلوية .و الرقم الهيدروحينى العالى بمصدر المياه ولذلك فمؤشر تقييم نوعية المياه يلعب دوراً رئيسيا فى اختيار نوع المادة المروبة المستخدمة فى معالجة المياه. كما أن الجرعة الزائدة او المنخفضة تؤدى إلى الاقلال من كفاءة عملية الازالة , ويتم التاكد من إنجاز هذه المهمة بعمل Jar test

## العوامل المؤثرة فى انجاز عملية التر ويب

يوجد عدة عوامل يجب اخذها فى الاعتبار لانجاز عملية التر ويب بكفاءة وهى كالتالى:

**١. طرق المزج**

يجب مزج المواد المروبة (الشبة) مع المياه بأسرع وقت (عدة ثوان)

حيث أن زمن هذا التفاعل قصير , ثم تتم عملية التفاعل بعد ذلك بعدة طرق منها

- المزج الهيدروليكى
- المزج الميكانيكى
- المزج بنظام الهواء المضغوط
- المزج بطريقة الحقن فى طلمبات المياه العكرة

لكى تتم عملية الترويب والتنديف فإنه يجب أن يلامس المروب كل اجزاء المواد العالقة ويحدث ذلك بعملية التقليب السريع

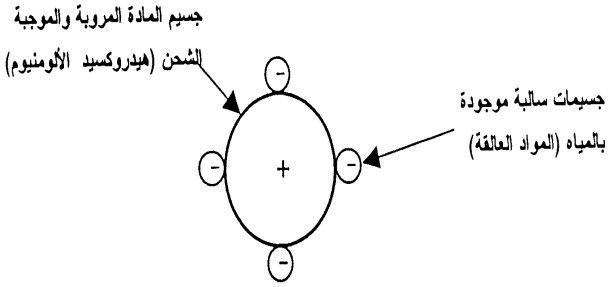
## ثانيا :- التنديف

هى عملية مزج بطئ للشبة تعمل على تجميع الحبيبات الصغيرة لتكوين حبيبات أكبر قابلة للترسيب ويستلزم ذلك عمل تقليب بطئ باستخدام الخلاطات الهيدروليكية والميكانيكية لإتاحة فرصة أكبر لعمل تلامس بين الندف الدقيقة لتلتصق ببعضها مشكلة ندف اكبر لترسب , وتتطلب هذه العملية زمن يتراوح من ١٥ :٢٠ دقيقة .

## وصف العملية

يتراوح الحجم الامثل للندف من (١ مم – ٣ مم) ويعتمد ذلك على نوع عملية الازالة المستخدمة سواء كان بنظام الترشيح المباشر أو الترشييح التقليدى ويجب الاخذ فى الاعتبار وقت المزج والطريقة الصحيحة للمزج وشكل أحواض المزج. فالمزج السريع لمدة كبيرة يجعل الندف المتكونة تتباعد عن بعضها البعض وبالتالى تتكون ندف ضعيفة ومفككة بعد ان كانت ملتصقة ببعضها. فالتنديف عملية أبطأ بكثير من الترويب لذلك فإن حوض التنديف يجب أن يكون أكبر نسبياً ونظراً لأن الندف هشة تماماً فلذلك يجب أن يجرى الخلط بطيئاً كما يجب أن تكون سرعة التصرف خلال الحوض بطيئاً بما يكفل عدم تفتيت جسيمات الندف .و تكسيرها وعلى ذلك يجب أن يكون حوض التنديف كبرا بما يكفى. لتهيئة مدة المكث الملائمة.

ويحمل هيدروكسيد الألومونيوم المتكون شحنة كهربائية موجبة، وبالتالى يحدث تجاذب بينه وبين المواد العالقة والغرويات الدقيقة والبكتريا والمواد الغروية والتى تحمل شحنة سالبة , فيحدث تعادل كهربائى، وتلتصق ببعضها مكونة حبيبات أكبر تسمى ندف، والتى تزداد حجماً وثقلا أثناء تحولها فى الماء نتيجة تماسك الندف الدقيقة مع بعضهاه مكونة ندفا أكبر واثقل , قابلة للترسيب بسرعة أكبر. وتسمى هذه العملية بالتندف



و كل جزء فى المليون ( جم/ م$^3$) من الشبه الصلبةيقلل قلوية الماء بنسة٠.٤٥ جزء فى المليون ويزيد ثانى أكسد الكربون بمقدار ٠.٤ جزء فى المليون

## تأثير إضافة جرعة شبة قدرها ٣٠ جم/م$^3$

| م | نوع التحليل | مياه عكرة | مياه مرشحة |
|---|---|---|---|
| ١ | القلوية (ppm) مقاسة ككربونات الكالسيوم | ١٨٠ | ١٦٦ |
| ٢ | ثانى أكسيد الكربون (ملجم/ لتر) | ١,٤ | ١٢,٦ |
| ٣ | الرقم الأيدروجينى pH | ٨,١ | ٧,٥ |
| ٤ | العكارة (NTU) | ١٨ | ٤,٦ |

## العوامل التى تؤثر فى عملية الترويب و التنديف

**تتاثر عملية الترويب و التنديف بعولمل مختلفة :-**

- تركيز الاس الهيدروجينى (pH) ولكل نوع من المواد المروبة مدى فى الرقم الهيدروجينى. فتتم عملية الترويب بأعلى كفاءة بالنسة للشبة عندما يكون الرقم الهيدروجينى من ٤.٥ إلى ٧.٥ . اما بالنسبة لكبريتات الحديدوز فتحدث عند رقم هيدروجينى أكبر من ٨.٥
- قلوية الماء ,حيث تتم عملية التر ويب اسرع مع القلوية العالية .
- ظروف خلط المادة المروبة، ويفضل أن يكون خلط المادة المروبة بسرعة وبتجاس فى كل حجم المياه الخام .
- نسبة العكارة , ويفضل ألا تقل عن نسبة معينة ,فإذا كانت قليلة جداً فأحياناً يتم اللجوء إلى إضافة مواد مساعدة لتكوين نواة تتجمع حولها الندف، ويؤدى ذلك إلى الإقلال من المادة المروبة .
- جرعة المادة المروبة , والمقصود بها عدد جرامات المادة المروبة التى يتم إضافتها للمتر المكعب من الماء الخاما ويعبر عنها بالجرام لكل متر مكعب . و بجزء فى

المليون وذلك لاتمام عملية التر ويب والتندف باعلى كفاءة ولتكوين ندف ذات اكبر كثافة. ويفضل تحديد جرعة المادة المروبة عن طريق التجارب العملية باستخدام اختبار تحديد الجرعات (Jar test)

**ويتناسب حجم الندف طردياً مع جرعة الشبة المضافة وذلك فى حدود معينة، وبالتالى يلزم تحديد جرعة الشبة التى تحقق تكوين الندف فى حجم رأس الدبوس

** ويتم تحديد جرعة الشبة الفعالة معملياً، بواسطة جهاز تحديد الجرعات الشبه (Jar test)

# الفصل الخامس

## تجهيز اعمال الترويب و التنديف

# الباب الخامس

# تجهيز اعمال الترويب والتنديف

**مقدمة:-**

للبدء فى أعمال الترويب والتندف يجب اولأ العناية بتداول وتخزين المواد المروبة ثم بعد ذلك البدء فى تحضير المحاليل الخاصة منها ليتم إضافتها للمياه متى تتم عملية التر ويب والتنديف بكفاءة.

**تخزين الكيماويات**

تتوفر المروبات فى شكل صلب او مسحوق أو حبيبات او سائل وغالبأ ما تكون السوائل اسهل فى الخلط والا ستعمال ويمكن شراء الكيماويات باحجام مختلفة تتر واح من اكيأس زنة ١٥ كيلو جرام وأوعية سعة ٢٠ لتر الى سيارات نقل صهريجية زنة ٥٠ طن أو أكثر .

ويجب دائمأ تخزين الكيماويات فى منطقة جافة عند درجة حرارة متوسطة ومنتظمة الى حد ماء حيث أن الرطوبة قد تسب تصلد او تجبس الكيماويات الجافة فمن الواجب تخزينها على طبالى خشبية نقالة للسماح بمرور الهواء أسفل الاكياس ومن المفضل ايضأ تخزين الكيماويات الجافة على أرضية تقبع فوق منطقة تحضير المحاليل حتى يمكن تغذيتها بالجاذبية الى المنطقة الخاصة بالتحضير وعلى أية حال فقد لا يكون ذلك عملياً فى المحطات الصغيرة والمتوسطة الحجم لأن حجم الكيماويات المستعملة قد لايعتبر إستثمارأ رئيمياً فى معدات تداول الكيماويات .

وتشمل المعدات المستعملة فى تداول الكيماويات العربات اليدوية أو الميكانيكية والأوناش العلوية والمصاعد والناقلات الميكانيكية .

## تحضير المحاليل

يتم عادة تخفيف المرويات فى محطة المعالجة الى محاليل ذات تركيز مناسب قبل إضافتها الى المياه الخام وغالباً ما يكون التركيز ١٠ % صلبة

وفى المحطات الصغيرة يمكن إجراء ذلك عن طريق تفريغ الأكياس يدوياً أو تفريغ الكيماويات السائلة عن طريق السيفون، أو الطلمبات الى أحواض التخفيف لتحضير المحاليل المغذية. ومن المهم أن يعتنى المشغل بخلط المادة الجافة أو السائلة خلطاً متساوياً ومتجانساً فى الماء, وفى المحطات الكبيرة يخصص مبنى كامل للتحضير الاتوماتيكى لمحاليل الترويب وقد يشمل مثل هذا المبنى على قوا ديس كبيرة وموازين وناقلات بسيور وأوناش وشبكات من المواسير وطلمبات لمعايرة الكيماويات وصهاريج للتخزين المؤ قت .

ويتكون محلول الشبة من محلول حمضى آكل يتفاعل مع المادة الحاوية له، لذلك يتم إنشاء أحواض تجهيز المحلول من المبانى أو الخرسانة ثم يتم تبطينها بمادة لا تتأثر بالأحماض مثل السيراميك او تدهن بمواد لا تتاثر بالأحماض مثل الإيبوكس والفيبرجلاس اوالألياف الزجاجية

## تحضير المحاليل المخففة من الشبة

يتم إضافة مأئة كيلو جرام من الشبة الصلبة (شيكارتين زنة الواحدة ٥٠ كجم ) لكل متر مكعب من المياه فى الحوض لتعطى تركيز ١٠ % أما في حالة الشبة السائلة ( ٥٠ %) ذات كثافة ١.٣٢ فيتم التخفيف الى ٢٠ % من وزنها ( ما يعادل ١٠ % صلبة)

مثال : حوض أبعاده ٢ × ٢× ٦ متر (إرتفاع)

سعة حوض التخفيف ٢٤ م$^{٣}$

كثافة الشبة ٥٠% = ١,٣٢

كتلة الشبة فى الحوض كصلبة (١٠٠%) = $\frac{٢٤ \times ١٠٠}{١٠٠٠}$ = ٢,٤٠٠ طن

كتلة الشبة فى الحوض كسائلة (٥٠%) = ٢,٤ × ٢ = ٤,٨ طن

حجم الشبة فى الحوض كسائلة (٥٠%) = $\frac{٤,٨}{١,٣٢}$ = ٣,٦٣ م$^{٣}$

أى يضاف ٣٦٣٦ لتر فى حوض سعة ٢٤ م٣ من الشبة (٥٠%) لتعطى تركيز ١٠% وهو يعادل = ٣٦٣٦ × ١,٣٢ = ٤,٨ طن.

| حجم محلول الشبة م$^{٣}$ | ارتفاع المياه | حجم المياه م$^{٣}$ | كمية الشبة/طن | حجم الشبة م$^{٣}$ |
|---|---|---|---|---|
| ٢٤,٠ | ٥,٠٩١ | ٢٠,٣٦٤ | ٤,٨ | ٣,٦٣٦ |
| ٢٢,٠ | ٤,٦٦٦ | ١٨,٦٦٧ | ٤,٤ | ٣,٣٣٣ |
| ٢٠,٠ | ٤,٢٤٢٥ | ١٦,٩٧٠ | ٤ | ٣,٠٣٠ |
| ١٨,٠ | ٣,٨١٨ | ١٥,٢٧٣ | ٣,٦ | ٢,٧٢٧ |
| ١٦,٠ | ٣,٣٩٤ | ١٣,٥٧٦ | ٣,٢ | ٢,٤٢٤ |
| ١٤,٠ | ٢,٩٦٩ | ١١,٨٧٩ | ٢,٨ | ٢,١٢١ |
| ١٢,٠ | ٢,٥٤٥ | ١٠,١٨٢ | ٢,٤ | ١,٨١٨ |
| ١٠ | ٢,١٢١ | ٨,٤٨٥ | ٢ | ١,٥١٥ |
| ٨,٠ | ١,٦٩٧ | ٦,٧٨٨ | ١,٦ | ١,٢١٢ |
| ٦,٠ | ١,٢٧٢ | ٥,٠٩١ | ١,٢ | ٠,٩٠٩ |
| ٤,٠ | ٠,٨٤٨ | ٣,٣٩٤ | ٠,٨ | ٠,٦٠٦ |

- جدول يوضح كمية وحجم الشبة السائلة اللازمة لتحضير محلول تركيزه يعادل ١٠% صلبة لإرتفاعات مختلفة من الحوض.

## تحضير محاليل من الشبه الصلبة و السائلة

يجب تخفيف الشبة الى تركيز مناسب قبل إضافتها الى الماه الخام فى أحواض, ويضاف الماء النقى اليها لتحضير المحلول كما يلى :

### أ. فى حالة الشبة الصلبة(شكاير ٥٠ كجم ):

بفرض ان سعة حوض الشبة ١٥٤ م$^3$ (١١ متر طول X٣.٥ عرض X ٤ م إرتفاع) فإنه يتم إسقاط ٣٠٨ شيكارة شبة صلبة فى الحوض للحصول على محلول بتركيز ١٠ % ( يتم اسقاط شيكارتين لكل ١ م$^3$ من حجم الحوض.

**ويتم إتباع الخطوات التالية لتجهيز المحلول المطلوب :**

١ . يتم إضافة مياه للحوض من المحبس وعندما يصل منسوب المياه لمستوى ريش الخلط يتم تشغيله لتقليب المحلول.

٢ . يغلق محبس إضافة المياه للحوض عند ملئه.

٣ . استمر فى التقليب حتى يتم إذابة كل الشبة.

٤ . إبلاغ المعمل لأخذ عينة فى اليوم التالى.

٥ . يتم السحب من حوض الشبة حتى يصل الى مستوى عوامة أقل منسوب

### ب . فى حالة الشبة السائلة( تركيز ٥٠ % من الخزان):

للحصول على محلول شبة تركيز ١٠ % يتم ملء الحوض بالمياه لأى إرتفاع نختاره طبقاً للجدول ثم يفتح محبس دخول الشبة السائلة بتركيز ٥٠ % من الخزان بالارتفاع المناظر له من الجدول مع التقليب بواسطة الخلاط للحصول على محلول شبة تركيز ١٠ %

## تحضير محلول شبة بتركيز ١٠% باستخدام شكاير الشبة الصلبة

| ارتفاع محلول الشبة ١٠% المطلوب تجهيزه (م) | حجم محلول الشبة ١٠% المطلوب تجهيزه (م$^3$) | عدد شكاير الشبة • |
|---|---|---|
| ٤,٠٠ | ١٥٤,٠٠ | ٣٠٨,٠٠ |
| ٣,٩٠ | ١٥٠,١٥ | ٣٠٠,٣٠ |
| ٣,٨٠ | ١٤٦,٣٠ | ٢٩٢,٦ |
| ٣,٧٠ | ١٤٢,٤٥ | ٢٨٤,٩ |
| ٣,٦٠ | ١٣٨,٦ | ٢٧٧,٢ |
| ٣,٥٠ | ١٣٤,٧٥ | ٢٦٩,٥ |
| ٣,٤٠ | ١٣٠,٩ | ٢٦١,٨ |
| ٣,٠ | ١٢٧,٠٥ | ٢٥٤,١ |
| ٣,٢ | ١٢٣,٢ | ٢٤٦,٤ |
| ٣,١ | ١١٩,٣٥ | ٢٣٨,٧ |
| ٣,٠ | ١١٥,٥ | ٢٣١,٠٠ |
| ٢,٩٠ | ١١١,٦٥ | ٢٢٣,٣٠ |
| ٢,٨٠ | ١٠٧,٨ | ٢١٥,٦ |
| ٢,٧٠ | ١٠٣,٩٥ | ٢٠٧,٩ |
| ٢,٦ | ١٠٠,١ | ٢٠٠,٢ |
| ٢,٥ | ٩٦,٢٥ | ١٩٢,٥ |
| ٢,٤ | ٩٢,٤ | ١٨٤,٨ |
| ٢,٣ | ٨٨,٥٥ | ١٧٧,١ |
| ٢,٢٠ | ٨٤,٧ | ١٦٩,٤ |
| ٢,١ | ٨٠,٨٥ | ١٦١,٧ |
| ٢,٠ | ٧٧,٠ | ١٥٤,٠ |
| ١,٩٠ | ٧٣,١٥ | ١٤٦,٣ |
| ١,٨ | ٦٩,٣ | ١٣٨,٦ |
| ١,٧ | ٦٥,٤٥ | ١٣٠,٩ |
| ١,٦ | ٦١,٦ | ١٢٣,٢ |

• وزن شيكارة الشبة الصلبة هو ٥٠ كجم

## تحضير محلول شبة يعادل تركيز ١٠ % صلبة باستخدام الشبة السائلة ذات تركيز ٥٠ %*

| ارتفاع محلول الشبة المطلوب تجهيزه (م) | ارتفاع المياه المضافة (م) | حجم المياه (م$^3$) | ارتفاع الشبة ٥٠% المضافة (م) | حجم الشبة ٥٠% (م$^3$) |
|---|---|---|---|---|
| ٤ | ٣,٣٩٤ | ١٣٠,٦٦٩ | ٠,٦٠٦ | ٢٣,٣٣٣ |
| ٣,٩ | ٣,٣١ | ١٢٧,٤٣٥ | ٠,٥٩٠ | ٢٢,٧٥ |
| ٣,٨ | ٣,٢٢٥ | ١٢٤,١٦٢ | ٠,٥٧٥ | ٢٢,١٦٦ |
| ٣,٧ | ٣,١٤ | ١٢٠,٨٩ | ٠,٥٦٠ | ٢١,٥٨٣ |
| ٣,٦ | ٣,٠٥٥ | ١١٧,٦١٧ | ٠,٥٤٥ | ٢٠,٩٩٩ |
| ٣,٥ | ٢,٩٧ | ١١٤,٣٤٥ | ٠,٥٣٠ | ٢٠,٤١٦ |
| ٣,٤ | ٢,٨٨٥ | ١١١,٠٧٢ | ٠,٥١٥ | ١٩,٨٣٣ |
| ٣,٣ | ٢,٨٠ | ١٠٧,٨ | ٠,٥٠٠ | ١٩,٢٤٩ |
| ٣,٢ | ٢,٧١٥ | ١٠٤,٥٢٧ | ٠,٤٨٥ | ١٨,٦٦٤ |
| ٣,١ | ٢,٦٣ | ١٠١,٢٥٥ | ٠,٤٧٠ | ١٨,٠٨٣ |
| ٣ | ٢,٥٤٥ | ٩٧,٩٨٢ | ٠,٤٥٥ | ١٧,٤٩٩ |
| ٢,٩ | ٢,٤٦١ | ٩٤,٧٤٨ | ٠,٤٣٩ | ١٦,٩١٦ |
| ٢,٨ | ٢,٣٧٦ | ٩١,٤٧٦ | ٠,٤٢٤ | ١٦,٣٣٣ |
| ٢,٧ | ٢,٢٩١ | ٨٨,٢٠٣ | ٠,٤٠٩ | ١٥,٧٤٩ |
| ٢,٦ | ٢,٢٠٦ | ٨٤,٩٣١ | ٠,٣٩٤ | ١٥,١٦٦ |
| ٢,٥ | ٢,١٢٢ | ٨١,٦٩٧ | ٠,٣٧٨ | ١٤,٥٨٣ |
| ٢,٤ | ٢,٠٣٦ | ٧٨,٣٨٦ | ٠,٣٦٤ | ١٣,٩٩٩ |
| ٢,٣ | ١,٩٥٢ | ٧٥,١٥٢ | ٠,٣٤٨ | ١٣,٤١٦ |
| ٢,٢ | ١,٨٦٧ | ٧١,٨٧٩ | ٠,٣٣٣ | ١٢,٨٣٣ |
| ٢,١ | ١,٧٨٢ | ٦٨,٦٠٧ | ٠,٣١٨ | ١٢,٢٤٩ |
| ٢ | ١,٦٩٧ | ٦٥,٣٣٤ | ٠,٣٠٣ | ١١,٦٦٦ |
| ١,٩ | ١,٦١٢ | ٦٢,٠٦٢ | ٠,٢٨٨ | ١١,٠٨٣ |

* كثافة الشبة السائلة الخام ١,٣٢ جم / سم$^3$ في هذا المثال

و يمكن تأكيد تركيز الشبه فى احواض الشبة عن طريق قياس كثافتها و مقارنة القيمة بالجدول الاتى :-

| الكثافة بالهيدروميتر | التركيز المقابل لها |
|---|---|
| ١.٠٣ | %١٢ |
| ١.٠٤ | %١٦ |
| ١.٠٥ | %٢٠ |
| ١.٠٦ | %٢٢ |
| ١.٠٧ | %٢٦ |
| ١.٠٨ | %٣٠ |
| ١.٠٩ | %٣٤ |
| ١.١ | %٣٨ |

# الفصل السادس

## اختبارات التحكم فى التشغيل

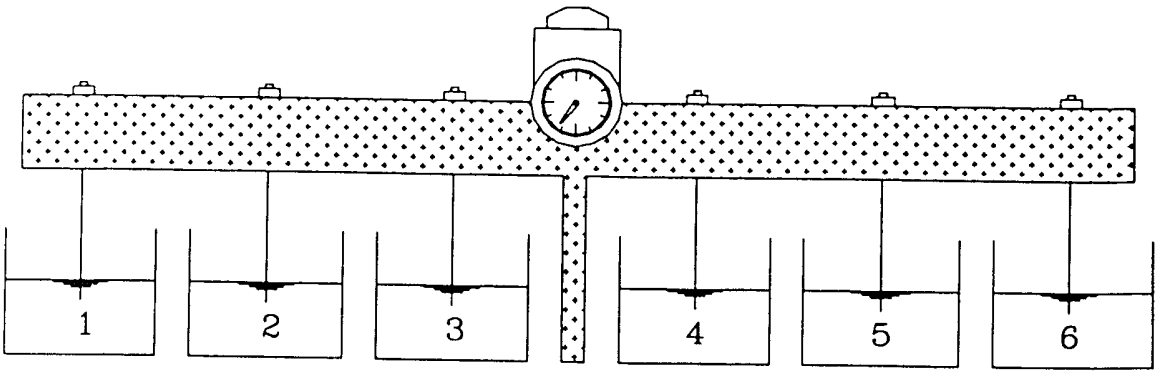
# الباب السادس

# اختبارات التحكم فى التشغيل

للبدء فى إجراءات التحكم فى التشغيل وتحديد إضافة الشبةو فإنه من الضرورى البدء أولاً فى إجراء تجربة اختبار تحديد الجرعات (Jar test) والتى يتم على ضوء نتائجها تحديد جرعة الشبة المطلوبة إضافتها للمياه .

## جهاز تحديد جرعات الشبه

يتكون الجهاز، كما هو مبين الشكل من مجموعه من الكؤوس الزجاجية و الذى يستخدم لتحديد جرعة الشبه الفعالة معمليا




ويوجد فى كل كاس قلاب صغير، ويتم إدارة هذه القلابات بواسطة عامود إدارة عن طريق تروس , ويتم إدارة العامود بواسطة محرك كهربانى صغير .

ويتم دوران القلابات عن طريق التروس بسرعة تماثل سرعة تحرك الماه فى المروقات ويتم التقليب السريع بسرعة ٨٠ –٣٠٠ لفة/ دقيقة لمدة (١٥ –١٢٠ ) ثانيه تخفض الصرعة إلى (٢٠– ٤٠ ) لفة/ دقيقة لمدة (١٥– ٤٥ ) دقيقة بعد إضافة الجرعات المختلفة .

# خطوات تجربة تحديد الجرعات

## ١. تجهيز المحلول

- **فى حالة الشبة السائلة:**

ضع ١٥.٦ ملليلتر من الشبة السائلة (كثافة نوعية ١.٣٢٥) فى قارورة سعة ١٠٠٠ مللى لتر وأكمل القارورة حتى علامة ١٠٠٠ بالماء للحصل على محلول شبة بتركيز ١٠٠٠٠مجم/ لتر ( ١ % )

## ملحوظة:

تغيير الكثافة النوعية للشبة السائلة حسب الشركة المنتجة.

- **فى حالة الشبة الصلبة:**

١. ضع ١٠ جم من الشبة الصلبة سلفات الألومونيوم($Al_2(SO_3)_3.18H_2O$) الى ٦٠٠ مللى لتر مياه فى قارورة سعتها ١٠٠٠ سم$^3$ وأكمل القارورة حتى علامة ١٠٠٠ سم$^3$ بالماء. لتحصل على محلول شبة بتركيز ١٠٠٠٠مجم/ لتر (١%)

٢. أجمع ٨ لتر عينة من المياه العكرة .

٣. ضع فى كل كاس من الكئوس الستة كمية مقدارها ١٠٠٠ مللى لتر من المياه العكرة .

٤. ضع الكئوس داخل الجهاز ثم اسقط ريش التقليب فى الكئوس وقلب الماء بسرعة تعادل سرعة المياه بأحواض المحطة .

٥. اضف بحرص وسريعاً بواسطة ماصة جرعة الكلور المقدمة لجميع الكنوس

6. استخدم الماصة فى إضافة جرعات شبه متزايدة بالتوالى لكئوس الاختبار الستة بحرص وبسرعة بقدر الامكان .

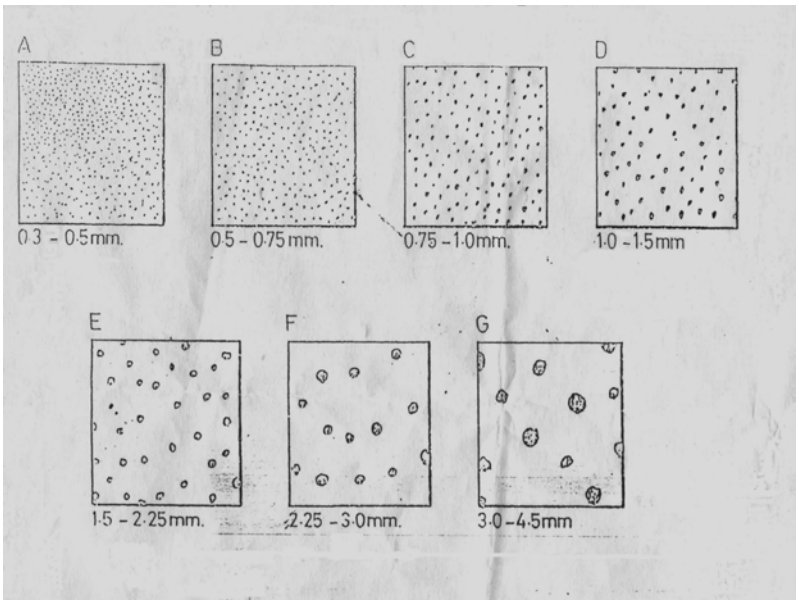
7-استمر فى التقليب السريع لمدة حوالى دقيقة واحدة من بداية اضافة الشبة .

8. قلل من سرعة التقليب لتصل إلى ٢٠ – ٤٠ لفه/ د ) واستمر فى تقليب المحلول من مدة تتراوح من ٢٠ إلى ٣٠ دقيقة .

لاحظ سرعة بداية تكون الندف فى كل كاس وشكلها وحجمها .

١٠. أوقف عملية التقليب ولاحظ ترسيب الندف بالكئوس لمدة ٥ – ١٠ دقيقة. وعادة تشير العينة ذات الغيوم إلى عدم كفاءة الترويب. أما ذات الترويب الجيدة فتبدو رائقة لأحتوائها على ندف مجمعة ثقيلة الوزن

** سجل النتائج بتصنيفها إلى : ضعيفة / مقبولة / جيدة / ممتازة .




١١. اسحب بواسطة سيفون على شكل حرف (S) فى نفس الوقت كمية المياه التى تمثل ارتفاع ٧ سم العلوية من الكاس وهى حوالى نصف لتر تقريباً للكاسات الستة، وجمعها فى عدد ستة مخابير لاجراء باقى الاختبارات عليها كما هو موضح فى الجدول

## نموذج تسجيل نتيجة فحص العينات

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| اسم محطة التنقية: | | | | | | | | |
| اختبار التنديف رقم: | | | | | | | التاريخ: | |
| الكيماويات المساعدة رقم (١): كبريتات الألومنيوم (الشبة) | | | | | | | الزمن: | |
| الكيماويات المساعدة رقم (٢): الكلور | | | | | | | | |
| الكيماويات المساعدة رقم (٣): | | | | | | | | |
| العملية | الوحدة | ١ | ٢ | ٣ | ٤ | ٥ | ٦ | ٧ |
| الشبة بتركيز ١٠ جم/ لتر | | | | | | | | |
| الكلور | ppm | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| جودة الندف: | | | | | | | | |
| الحجم | | | | | | | | |
| الكمية | مل | | | | | | | |
| سرعة الترسيب | | | | | | | | |
| عكارة المياه المروقة | NTU | | | | | | | |
| الطحالب | عدد/ مل | | | | | | | |
| الرقم الهيدروجينى | | | | | | | | |
| الكلور المتبقى | PPM | | | | | | | |
| الروبة %: | | | | | | | | |
| الكمية | | | | | | | | |
| معامل تماسك الروبة | | | | | | | | |
| المياه المرشحة | | | | | | | | |
| العسر | ملجم/ لتر | | | | | | | |
| القلوية | ملجم/ لتر | | | | | | | |
| المواد العضوية | | | | | | | | |
| الألومنيوم المتبقى | | | | | | | | |
| الرقم الهيدروجينى | | | | | | | | |
| العكارة | NTU | | | | | | | |
| أفضل النتائج: | | | | | | | | |
| المادة المساعدة | | PPM | المادة المساعدة | | | | | PPM |
| كبريتات الألومنيوم (الشبة) | | | | | | | | |
| كلور | | | | | | | | |

PPM : جزء فى المليون

## الحسابات اللازمة لتجربة JAR TEST

١. تعيين قيمة الG-VALUE للمزج لسريع وتحويلها الى لفة/دقيقة باستخدام الجدول التالى

٢. اذا تم استخدام المأخذ بعد حقن الكلور لابد من تحديد الفترة الزمنية فى المحطة بين اضافة الكلور والشبة

٣. يتم تعيين فترة المكث فى المزج السرع

٤. يتم حساب فترة المكث فى احواض التنديف

فترة المكث= الحجم/التصرف

٥. يتم حساب قيمة ال G-value للتنديف

٦. يتم حساب الحمل السطحى

الحمل السطحى=التصرف/مساحة السطح

**مع مراعاة** ان تكون مياة المأخذ متساوية فى الكمية والنوعية فى الكاسات وذلك بالتقليب المستمر للمأخذ بين اضافة كل كاس

١. يتم تحضير محلول الشبة وسحب المياة العكرة فى نفس يوم التجربة

٢. يتم قياس العناصر المطلوبة لمياة المأخذ

٣. يتم تحضير الكاسات والادوات التى سيتم سحب العينات بها بعد الترسيب للقياس

**علاقة ال G –Value بعدد اللفات للقلاب السريع**

| م | G-VALUE (S-1) | عدد اللفات rpm |
|---|---|---|
| ١ | 3.3 | 10 |
| ٢ | 9.5 | 20 |
| ٣ | 18 | 30 |
| ٤ | 27 | 40 |
| ٥ | 38 | 50 |
| ٦ | 61 | 70 |
| ٧ | 105.5 | 100 |
| ٨ | 200 | 150 |
| ٩ | 300 | 200 |
| ١٠ | 600 | 300 |
| ١١ | 870 | 400 |
| ١٢ | 1203 | 500 |

## وتوجد عدة عوامل هامة فى تقييم نتائج اختبار تحديد الجرعات :

- معدل تكوين الندف
- شكل جزيئات الندف
- شفافية المياه تحت بين الندف
- حجم هذه الندف
- كمية الندف المتكونة
- معدل ترسيب الندف
- شفافية المياه للطبقة الرائقة

وتختلف تركيزات الشبة المستخدمة فى اختبار الكئوس تبعاً لاختلاف درجة عكارة المياه الخام أو بالتالى كمية الجرعة المطلوبه للمعالجة كما هو موضح بالجدول .

**تركيز المواد الكيماوية (الشبة) المستخدمة فى إختبار ( Jar Test)**

| الجرعة المطلوبة مللجم / لتر | عدد الجرامات /لتر (التحضير) | ١ مللى من المضاف لكل كأس يساوى | تركيز المحلول % |
|---|---|---|---|
| ١- ١٠ | ١ جم / لتر | ١ مللجم / لتر | ٠,١ % |
| ١٠-٥٠ | ١٠جم / لتر | ١٠ مللجم / لتر | ١% |
| ٥٠-٥٠٠ | ١٠٠جم / لتر | ١٠٠مللجم / لتر | ١٠% |

## اختبارات التحكم فى التشغيل

توجد خمسة اختبارات تساعد على التحكم فى عملية الترويب/التندف وهى كالآتى:

- **اختيار تحديد الجرعات (Jar test )**
- **اختبار الرقم الهيدروجينى (pH)**
- **قابلية الترشيح**
- **جهد زيتا Zp**
- **الشفافية (أو العكارة)**

## الرقم الهيدروجينى

يجب معرفة القائم على التشغيل لدرجة التاين الهيدروجينى المثلى لكل نوع من المرويات ويعطى أفضل نتائج تساعده على اختيار المروب الملانم أو يعمل على تصحيحها وضبطها قبل إجراء عملية التر ويب. كما أن قياس الرقم الهيدروجينى (pH)فى عينات المروق تعطى الدلائل الصحيحة على جودة الترويب .

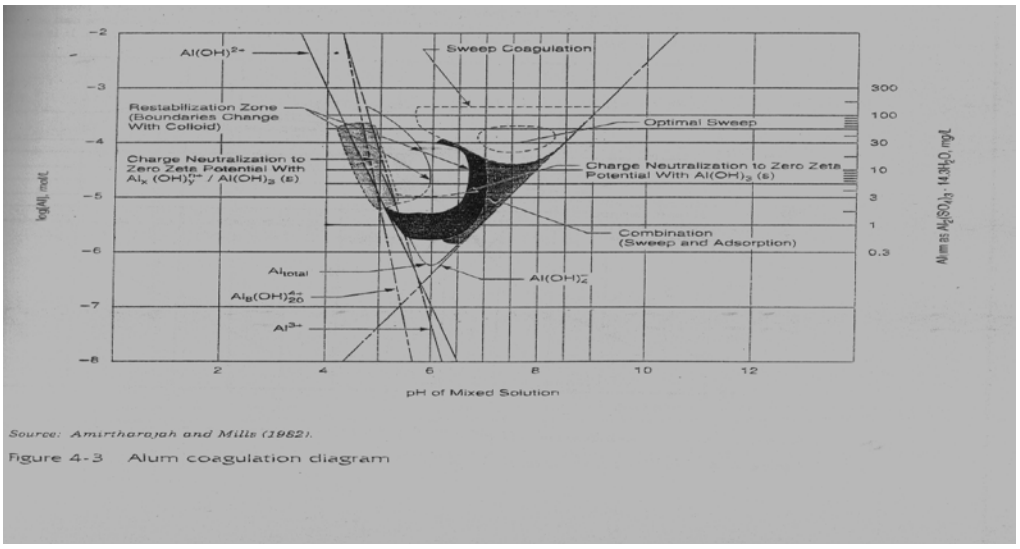


Figure 4-3 Alum coagulation diagram

الشكل السابق هو diagram تم تصميمة لتسهيل فهم عمل الشبة ومن الشكل يتضح الاتى:

- الاس الهيدروجينى والجرعة هما العاملان الرئيسيان المتحكمان فى عمل الشبة مع درجة الحرارة والمواد العضوية والتى تزيد من ذوبان الالمونيوم
- المنطقة خارج حدود المثلث هى منطقة ذوبان للالمونيوم ولا يحدث اى ترويب او تنديف
- المنطقة داخل حدود المثلث هى منطقة عمل الشبة
- مثال :جرعة ١٠٠ جم/م٣ لا تعمل عند pH اعلى من ٩.٥ او اقل من ٤.٥ لتكون الهيدروكسيدات الذائبة بينما تبدأ العمل عند pH اقل من ٩ واعلى من ٤.٥حيث يبدأ تكون الهيدروكسيدات القابلة للترسيب
- بينما جرعة ٣جم/م٣ تبدأ العمل من pH من ٧ – ٥
- كل 1 مجم/لتر شبة صلبة يحتوى على ٠.٠٩ مجم/لتر +AL3(تركيز ١٦–١٧%)

- كل 1 مجم/لتر شبة سائلة يحتوى على ٠.٠٤٥ مجم/لت+3 AL(تركيز ٨%)
- نقطة عدم الذوبان للالمنيوم تحدث عند pH 6-6.2 صيفا وعند pH 6.5-7 شتاءا
- الشبة حمضية وعند اضافتها للماء تؤدى الى انخفاض ال pH

## جهد زيتا (Zp)

يستعمل اختبار جهد زيتا للمساعدة على تعيين أفضل د رجة تاين هيدروجينى وأفضل جرعة للمرويات مثل الألومونيوم +++Al لذا يجب مراقبة جهد زيتا (Zp)ليتراوح فى مدى معلوم لانتاج أقل عكارة ويتفاوت هذا المدى من محطة إلى محطة ويتراوح غالباً من 4 – 10 مللى فولت

**درجة الترويب فى نطاقات مختلفة لجهد زيتا**

| متوسط جهد زيتا | درجة الترويب |
|---|---|
| (٣٠) – (صفر) | الحد الأقصى |
| (-١) – (-٤) | ممتازة |
| (-٥) – (-١٠) | متوسطة |
| (-١١) – (-٢٠) | ضعيفة |
| (-٢١) – (-٣٠) | معدومة تقريباً |

## اختبار قابلية الترشيح

يستخدم اختبار قابلية الترشيح كطريقة لاختبار مدى الفاعلية التى يمكن بها ترشيح مياه مروبة، ويقيس الاختبار مقدار المياه المرشحة فى مدة معينة ويجرى عادة بترشيح كمية صغيرة من مياه المحطة المروبة والمندفة فعلا قبل الترسيب فى مرشح قطره ١٥ سم( مرشح استرشادى) ويحتوى على نفس نوع وعمق وسط الترشيح المستعمل فى مرشحات المحطة ويزود هذا المرشح بجهاز لقياس العكاره ليسجل عكارة المياه الخارجية حيث تجرى قياسات مستمرة لعكارة المياه المرشحة ومن مميزات هذا الاسلوب فى التحكم فى الشغيل ما يلى :

- أن الماء المستعمل فى هذا الاختبار هو الماء المروب فعلا بالمحطة وليس لعينة مماثلة يتم تحضيرها فى المعمل.
- أن الماء عند أية نقطة يستغرق نحو ١٠ دقائق للمرور خلال المرشح

ولما كانت مدة المكث فى حوض الترسيب بحجمه الكامل وفى المرشحات الفعلية تستغرق نحو ساعتين إلى ثلاث ساعات لذلك فإن التغيرات فى جرعات المروب يمكن أن تجرى بناء على أداء المرشح قبل حدوث تدهور خطير فيه . وبصفة عامة فإن إجراء اختبارات **(Jar test)** واختبار **(pH)** ودرجة العكارة والشفافية (اختبارات التحكم فى التشغيل ) تعطى دلالة جيدة عن النتائج المتوقعة إلا أن تشغيل المحطة على الواقع قد لا يتطابق مع هذه النتائج , لذا يجب مراقبة الأداء الفعلى من أجل:

- ملائمة الجرعة المناسبة لنوعية المياه المستخدمة.
- ملائمة الخلط السريع وحسن انشار المادة المروبة.
- ملائمة عملية التنديف البطىء.
- خزانات المياه المروبة والمرشحة.

ويعتمد التنديف الناجح على الخلط السريع والتام ورغم أن التر ويب يحدث فى أقل من ثانية واحدة إلا أن حوض الخلط قد يهيئ مدة مكث تصل إلى ٦٠ ثانية ويجب أن يكون الخلط عن طريق عملية الدوامات التى تحدث فى حركة المياه حتى ينشر المروب فى كل حوض الترويب .

ويرى البعض أنه خلال العشر ثوان الاولى من نقطة الإضافة يجب أن يخلط المروب اختلاطاً تاماً مع كل قطرة مياه لبدء تكوين ندف فعالة وإلا فإن كفاءة العملية كلها تهبط ويتضح تاثير معظم الاستعمال الخاطئ للمروب عند هذه النقطة. ويمكن تصحيح هذه المشاكل إما بتعديل نقطة الإضافة أو طريقة الخلط .

## ضبط جرعة الشبه

يتم ضبط جرعة الشبة طبقاً لنسة العكارة فى المياه ويعاد ضبط جرعة الشبة كلما تغيرت العكارة فى المياه بالتنسيق بين مشرف التشغيل وكيميائى المعمل ولا يتم عملية ضبط الجرعة مرة واحدة بل يلزم المتابعة المستمرة لها حسب توجيهات المعمل لتقليل استهلاكها مع المحافظة على أعلى نسة جودة للمياه , ولو أمكن مثلا تقليل نسبة استهلاك الشبة, فإن ذلك يوفر كثيرأمن تكاليف التشغيل

## طريقة حساب جرعة الشبه

بفرض أن طلمبة الشبة تعطى ٥١٤ جالون/ ساعة عند مشوار ١٠٠ % المكبس ولنفرض أن المعمل حدد جرعة الشبة ٣٠ جم/م$^{٣}$ لمروق يعطى ٥٠٠٠ م$^{٣}$/ ساعة

.: كمية الشبه الصلبة المطلوبة = ٥٠٠٠ X ٣٠ = 15000 جم / ساعة و لكن حيث الشبة المستخدمة سائلة بتركيز ١٠ % ( ١٠ جم شبه صلبة / ١٠ مل ماء)

∴ كمية الشبة السائلة المطلوبة = ١٥٠٠٠٠ × ١٠٠ / ١٠

= ١٥٠٠٠٠٠ مل/ ساعة

= ١,٥ م$^{٣}$ / ساعة

∴ مكبس الطلمبة يعطى ٥١٤ جالون/ ساعة عند مشوار ١٠٠ %

= ٥١٤ × ٣,٨ لتر/ ساعة

= ١,٩٥ م$^{٣}$ / ساعة

∴ لكى تحصل على جرعة ١,٥ م$^{٣}$/ ساعة فإنه يلزم ضبط المشوار بنسبة

= ١٠٠ x ١,٥ / ١,٩٥ = ٧٦%

وحيث أن أقصى مشوار للمكبس مقسم إلى ١٠٠ قسم (علامة).

∴ يتم ضبط مشوار للمكبس على العلامة (٧٦).

ملحوظه: جالون أمريكى = ٣.٨ لنر

## خطوات حساب جرعة الطلمبات

لحساب كمية الشبة التى يجب إضافتها فى الساعة إلى المياه العكرة باستخدام طلمبة الحقن فإنه يجب تحديد فتحة تصرف الطلمبة كالآتى:

فتحة تصريف الطلمبة = (كمية المياه العكرة (م$^{٣}$/س) × الجرعة جم/م$^{٣}$ × ١٠٠) / (تركيز الشبة (جم/لتر) × سعة الطلمبة (أقصى تصريف) (لتر/س))

وبفرض أن كمية المياه العكرة التى تدخل بيارة التوزيع تساوى ٢٠٠٠ م$^{٣}$/ساعة وتركيز محلول الشبة المعد للغستخدام ١٠% والجرعة المطلوبة ٢٥جم / م$^{٣}$، سعة طلمبة الشبة ١٢٠٠ لتر/ س.

∴ تكون كمية الشبة المطلوبة فى الساعة = $\frac{٢٥ \times ٢٠٠٠}{١٠}$ x ١٠٠

= ٥٠٠,٠٠٠ مل = ٥٠٠ لتر

وبالتالى فإنه يتم ضبط الطلمبة على فتحة مقدارها = $\frac{١٠٠ \times ٥٠٠}{١٢٠٠}$

= ٤١,٦% تقريباً

أى أن الطلمبة تضبط على ٤١,٦% من سعتها لتعطى ٥٠٠ لتر/ ساعة بجرعة قدرها ٢٥ جم/ م$^{٣}$ لكل ٢٠٠٠ م$^{٣}$ مياه.

---

$Al_2 (SO_4)3 + 3Ca (HCO_3)_2$ ------- $2Al (OH)_3 + 3CaSO_4 + 6CO_2$

$Al_2 (SO_4)3 + 3 NaCO_3 + 3H_2O$------- $2Al2 (OH)_3 + 3Na2SO_4 + 3CO_2$

$Al2 (SO4)3 + 3Ca (OH)_2$ ---------- $2Al (OH)_3 + 3CaSO_4$

من المعادلة السابقة يتضح ان

- كل ١ مجم/لتر شبة صلبة يقلل القلوية بمقدار ٠.٤٥ مجم/لتر as CaCO3
- كل ١ مجم/لتر شبة سائلة يقلل القلوية بمقدار ٠.٢٢٥ مجم/لتر as CaCO3
- يتم قياس فرق القلوية بين العكرة والمزج السريع او بداية حوض التنديف

# الفصل السابع

## المواصفات القياسية المصرية

## ١٩٨٢-٤

# الباب السابع

## المواصفات القاسيه المصرية ٤- ١٩٨٢

## كبريتات الالمونيوم ( الشبه الصلبة) المستعملة فى عمليات معالجة مياه الشرب

**مقدمة**

هذه المواصفات تلغى وتحل محل المواصفات القياسية المصرية رقم ٤ لسنة ١٩٥٨ التى سبق قيدها ونشرها بالسجل الرسمى فى ١٨٥٨/١٠/٣.

تختص هذه المواصفات القياسية بمواصفات الشبة السائلة المستخدمة فى عمليات معالجة مياه الشرب ، وتحضر بالتعامل الكيميائي بين حمض الكبريتيك مع أحد الخامات الغنية بالألومنيوم مثل الكاولين أو غيره.

تحدد هذه المواصفات القياسية خواص كبريتات الألومونيوم (الشبه) المستعملة فى تنقية مياه الشرب التى تنطبق عليها الصيغة الجزئية ($Al_2(SO_4)_3.18H_2O$) وتحضر بالتعامل الكيمائى بين حمض الكبريتيك مع أحد الخامات الغنية بالألومنيوم مثل البوكسيت (أكسيد الألومنيوم )

**٢. الوصف**

يجب أن تكون كبريتات الألومنيوم (الشبه) المستعملة فى تنقية مياه الشرب.

- من النوع النقى
- تكون على هيئة قوالب أو قطع صغيرة أو حبيبات
- تكون خالية من المواد الضارة (تحدد ها المعامل المركزية بوزارة الصحة)
- شفافة بحيث تسمح بقراءة أجهزة القياس بسهولة.

**٣ . الخواص الطبيعية و الكيميائية :-**

- لاتزيد نسبة المواد غير القابلة للذوبان فى الماء على ٠.٥
- لاتقل نسبة أكسيد الألومنيوم عن ١٥.٣ %
- لا تزيد نسة الحديديك ٠.٩ %

- لا يزيد الزرنيخ على ٥٠ جزء فى المليون

## ٤ سحب العينات

تسحب عينات عشوائية متجانسة وممثلة بمعرفة الجهات المختصة من كل خزان من خزانات التخزين الموجودة لدى المنتج والمستهلك على السواء وكذا من صهارج شاحنات النقل وكذلك عند الا ستخدام لدى المستهلك .

### حجم العينات

- تسحب العينات بحجم لايقل عن ٢ لتر وتمزج جيدا ويعبأ منها ثلاثة عبوات بحجم لا يقل عن٢/١ لتر فى وعاء زجاجى محكم الغلق غير منفذ للهواء ثم تحرز وتختم بالشمع الاحمر .
- يكتب على كل وعاء بخط واضح التاريخ الذى أخذت فيه العينه ورقم الرسالة وتوقيع المشترى أو المورد أو مندوبيهما .
- ترسل عبوة للمعمل الكيميانى المختص لاجراء التحليل اللازمة عليها .
- تحفظ العبوتان الباقيتان واحدة طرف المورد والأخرى طرف المشترى .

### النقل والتخزين

تنقل الشبة السائلة فى صهاريج قياسية مناسبة إلى مكان الا ستخدام وتخزن فى خزانات مناسبة لا تتأثر بالشب ولاتؤثر فيها .

### اخذ العينات

تقسيم العينة المرسلة بطريقة التربيع ثم يؤخذ منها حوالى ١٠٠ جرام وتطحن ثم تجرى عليها الفحوص اللازمة .

**اولا:**

أ. اذا كانت كمية الرسالة لا تزيد على ٢٠٠٠٠ عبوة فيؤخذ منها عدداً من العبوات طبق للجدول الأتى:

| حجم الرسالة بالعدد | | حجم العينة بالعدد |
|---|---|---|
| من | إلى | |
| ٢ | ٨ | ٢ |
| ٩ | ٢٧ | ٣ |
| ٢٨ | ٦٤ | ٤ |
| ٦٥ | ١٢٥ | ٥ |
| ١٢٦ | ٢١٦ | ٦ |
| ٢١٧ | ٣٤٣ | ٧ |
| ٣٤٤ | ٥١٢ | ٨ |
| ٥١٣ | ٦٢٩ | ٩ |
| ٧٣٠ | ١٠٠٠ | ١٠ |
| ١٠٠٠ | ١٠٠٠٠ | ١١ |
| ١٠٠٠٠ | ٢٠٠٠٠ | ١٢ |

ويراعى عند أخذ العبوات أن تكون من مواضع مختلفة من الكمية .

ب. اذا كانت الكمية أكثر من ٢٠٠٠٠ عبوة تقسم الكمية إلى مجموعات كل مجموعة ١٠٠٠ عبوة ويؤخذ من كل منها عشر عبوات (١%) وتجمع العبوات المختارة ويؤخذ منها ٥ % من عددها حسبما اتفق فتكون هذه العبوات هى التى تمثل الكمية .

## ثانيا:-

يؤخذ من كل عبوة حوالى نصف كيلو جرام وذلك بعمل ثقب أو فتحة فى العبوة ويراعى اختيار المواضع بحيث تكون غير متشابهة أو متماثلة فى العبوات المختلفة ثم تجمع العينات جميعها وتخلط مع بعضها خلط جيدا ثم تجزأ بطريقة التربيع حتى نحصل على العينة ممثلة وزنها حوالى ثلاثة كيلو جرامات .

## ثالثا:-

تقسم العينة الممثلة إلى ثلاث عينات تزن الواحدة منها حوالى واحد كيلو جرام وتوضع كل منها داخل وعاء جاف نظيف من البلاستك أو الزجاج ثم تقفل جميعها قفلا جيدا أو تختم بالشمع الاحمر ويكتب عليها بخط واضح التاريخ الذى أخذت فيه العينة رقم الرسالة وتوقيع المكلف بأخذها وترسل أحداها للمعمل الكيميائى لاجراء التحليل اللازمة عليها ويحتفظ بالثانية لدى المشترى والثالثة لدى المورد وذلك لاحتمال طلب اعادة التحليل من أحد طرفى التعاقد.

## ٥. طرق الاختبار

### طريقة تعين المواد غير القابلة للزوبان فى الماء

يوزن بدقة حوالى ٢٠ جم من العينة وتوضع فى كأس من الزجاج وتذاب فى حوالى ٢٠٠مللي لتر من الماء المقطر الساخن وتقلب لبضع دقائق ثم ترشح خلال بوتقة من الزجاج الملبد رقم ٤ أو بوتقة الالندوم موزونة وزنا دقيقا بعد تجفيفها عند درجة حرارة ١٠٠–١٠٥ مْ إلى أن يثبت الوزن وتحسب النسبة المئوية للمواد غير القابلة للذوبان فى الماء من العلاقة الاتية:

$$\text{النسبة المئوية} = \text{و}_{١} \times \frac{١٠٠}{\text{و}}$$

**حيث:**

و ١ = وزن المتبقى فى البوتقة

و = وزن العينة بالجرام

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### طريقة تعين أكسيد الالمونيوم و الحديد:

ينقل ناتج الترشيح من العملية السابقة بعد أن يبرد لحرارة المعمل إلى قنينة قياس سعتها ١ لتر ثم تكمل بالماء المقطر إلى العلا مة وبواسطة ماصة ٢٥ ملليلتر لتوضع فى كأس من الزجاج سعة ٤٠٠ ملليلتر وتخفف بالماء المقطر ليصير حجمها حوالى ٢٠٠ ملليلتر ثم يضاف اليه حوالى ٥ جم من كلوريد النوشادر.

ويقلب المحلول جيدا حتى الإذابة وبعد ذلك يضاف ١ ملليلتر من حمض الهيدركلوريك المركز وأربع نقط من حمض النيتريك المركز ويقلب المحلول ببط ء لبضع دقائق وذلك لأكسدة الحديد ثم يضاف نقطتان من دليل أحمر الميثيل .

يضاف محلول هيدروكسيد النشادر المخفف بنسبة ١:١ تدريجيا مع التقليب المستمر إلى أن يبدأ الراسب فى الظهور ويستمر فى إضافة النشادر نقطة فنقطة حتى يظهر اللون الأصفر. تغلى محتويات الكأس لمدة دقيقة واحدة يتحول خلالها اللون الأصفر إلى اللون الأحمر الوردى ويرشح المحلول فورا باستعمال ورق ترشيح من النوع سريع الرشح مثل

(ورق واتمان ٤١) مع مداومة غسل الراسب على ورق الترشيح بمحلول ٢ فى المائة من كلوريد النشادر الساخن إلى أن يتخلص تماما من الكبريتات من ناتج الترشيح .

ويجب مراعاة عدم زيادة مدة الغلى عن اللازم أو التأخير فى بدء عملية الترشيح. تجفف ورقة الترشيح بما عليها من راسب بعد ذلك فى فرن هوائى درجة حرارته ١٠٠ مْ ثم تنقل إلى بوتقة من البلاتين موزونة وزنا دقيقا وتحرق با ستعمال اللهب العادى اولا ثم تنقل بعد تمام حرق ورقة الترشيح إلى فرن درجة حرارته حوالى ١٠٠٠ مْ أو با ستعمال البورى لمدة ساعة ثم تنقل البوتقة إلى المجفف وتنزل فيه حتى متبرد .

توزن وتعاد إلى الفرن والمجفف مع تكرار العملية إلى أن يثبت الوزن:-

تحسب النسة المئوية لهذين الا كسيدين من العلاقة الاتية :

$$\text{النسبة المئوية} = \text{و} \times \frac{١٠٠}{\text{و}} \times \frac{١٠٠}{٢٥}$$

و١ = وزن الراسب المتكون

و = وزن العينة بالجرامات

## طريقة تعيين أكسيد الحديد

يحضر محلول قياس يحتوى على ٠.١ جم من أكسيد الحديديك من اللتر وذلك بأذابة ٠.٤٩ جم من كبريتات الحديدوز النشادرى فى ٥٠ ملليلتر من الماء المقطر و ٢٠ ملليلتر من حمص الكبريتك المركز ويسخن المحلول تسخينا هينا مع إضافة برمنجنات البوتاسيوم ( ٠.١ ع نقطة فنقطة إلى أن يتم أكسدة الحديد ويعرف ذلك عند ظهور لون وردى خفيف لا يزول بعد دقيقة واحدة ثم ينقل المحلول إلى قنينة قياس سعتها لتر. ويضاف من الماء المقطر الكمية المناسبة إلى العلامة ويصير كل١ ملليلتر من هذا المحلول محتويا على ٠.١ملليجرام من أكسيد الحديدك ( ٠.٠٠٠١ جرام أكسيد الحديدك ) وباستعمال طريقة التقدير المقارن بالألوان المعروفة يمكن تقدير كمية الحديد وذلك بأخذ ١ ملليلتر من ناتج الترشيح المذكور فى صدر العملية السابقة وتوضع فى انبوبة نسلر وتخفف إلى ٥ ملليلتر بالماء ويضاف اليها ٢.٥ ملليلتر من حمض الهيدروكلوريك

المخفف بنسبة ٣:١ ثم نضع نقط من محلول برمنجنات البوتاسيوم حتى يصير اللون ورديا لمدة ٥ دقائق ثم يكمل بالماء المقطر إلى ٤٥ ملليلتر.

ثم يضاف ٥ ملليلتر من محلول ثيوسيانات البوتاسيوم وفى أنابيب نسلر أخرى مشابهة لها تماما فى السعة والقطر يوضع ٠.٢ , ٠.٤ ٠.٦ , ٠.٨, ١.٠ , ملليلتر .. الخ من محلول أكسيد الحديديك القياسى السابق شرحة فى صدر هذا البند ويضاف اليها نفس نسب الكواشف وتكمل جميعها إلى ٠ ٥ ملليلتر بالماء المقطر وترج محتويات الأنابيب رجأ جيداً وتنقل الأنبوبة التى بها العينة إلى المكان المخصص لها فى جهاز قياس الالوان ثم تنقل إلى الجزء الاخر المخصص للمحاول القياسى الانابيب المذكورة فيما قبل لانتخاب الواحدة منها التى يتماثل لونها مع لونه الانبوبة التى بها العينة .

**وتحسب النسبة المئوية لاكسيد الحديديك فى العينة من العلاقة الاتية:**

$$\text{النسبة المئوية} = ح \times \frac{١}{١٠٠٠٠} \times \frac{١٠٠٠٠}{و} \times ١٠٠$$

**حيث:**

ح = حجم محلول الحديد القياسى بالملليلترات فى الأنبوبة المنتخبة

و = وزن العينة بالجرامات

وبطرح هذه النتيجة من النسبة المئوية لاكسيدى الالومنيوم والحديد تنتج النسة المئوية لاكسيد الالومنيوم على حدة .

# الفصل الثامن

## تطهير المياه

# الباب الثامن

# تطهير المياه

يعتبر الماء ثانى أهم العناصر الضرورية للانسان بعد الأكسجين الذي نستنشقه من الهواء. وكما أن الماء لازم لاستمرار الحياة فقد يكون سببا في القضاء عليها إذا استعمل ملوثا بجراثيم الأمراض منل التيفود والدوسنتاريا والكوليرا أو غيرها .

والاهتمام بالماء وما ينقله من أمراض ليس وليد العصر الحديث فلقد أوصى به بقراط ( اله الطب عند القدماء)

بغلي الماء الذى يستعمل للشرب كما ظهر واضحا على اللوحات الأثرية التى خلفا المصريين القدماء أنهم كانوا يقومون بغلى الماء قبل استخدامه للشرب .

ولقد أصبح من الصعب الحصول على مصدر المياه الآمن نظرا لأن بعض المدن تستعمل المجارى المائية المجاورة لها سواء كانت – انهارا أو بحارا أو مصارف – أو بحيرات لصرف مخلفاتها السائلة عليها سواء قبل أو بعد علاجها جزئيا أو كليا , لذلك فان من واجب الحكومات أن تراقب مل هذه الحالات وعليها ايجاد الحلول اللازمة لتحفظ للمجرى المائي سلامته من أي تلوث قد يضر بالصحة العامة أو يؤذى الثروة المائية .

يتضح مما سبق إن تطهير مياه الشرب أمر حيوي وضروري حرصا علي صحة المواطنين وضمانا لسلامتهم. وعملية التطهير في معالجة المياه تهدف في المقام الأول إلى التخلص من كل أنواع الكائنات الحية الدقيقة .

## عملية تطهير المياه

عملية تطهير المياه هى أهم خطوة من خطوات معالجة المياه فهى التى تضفى عليه صفة الصلاحية وتؤمنه ضد الأمراض المعدية المنقولة عن طريقه وتتم عملية تطهير المياه بطرق متعددة منها:–

## المعالجة الحرارية

وهي أول طريقة تم اكتشافها منذ سنوات عديدة ويتم فيها غليان الماء، باستخدام الغليات، و لا تستخدم هذه الطريقة في إنتاج كميات كبيرة من المياه نظرا لارتفاع تكلفة الطاقة المطلوبة لها وإن كانت تستخدم في تحلية المياه المالحة .

## المعالجة الإشعاعية

وتتم عن طريق مرور المياه علي لمبة تنتج أشعة فوق بنفسجية ويجب أن تكون المياه خالية من العكارة وقريبة جدا من اللمبه المشعة لذا لا تستخدم هذه الطريقة إلا فى إنتاج الكميات الصغيرة من المياه

## المعالجة الكيماوية

يمكن استخدام العديد من المواد الكيمائية فى تطهير المياه ومنها: الكلور (لكلورة ), ثانى أكسيد الكلورو برمنجانات البوتاسيوم ، الأوزون الجير, البروم, اليود.

## الشروط الواجب توافرها فى المواد المطهرة :

١. أن تكون قاتلة للجراثيم ولا تؤثر على صحة الإنسان.
٢. أن تكون رخيصة الثمن ومتوفرة محليا أو يسهل استيرادها طوال العام
٣. أن يكون استعمالها سهل ومأمونا.
٤. أن يكون تخزينها سهل وآمنأ.

## التطهير بالكلور

عملية إضافة الكلور من أكثر الطرق انتشارا في عمليات تطهير المياه وقد كان لظهور عملية لكلوره في أواخر القرن الماضي اثر كبير في القضاء بدرجة كبيرة علي الأمراض التي ينقلها الماء وذلك بأقل التكاليف وابسط المعدات واقل عدد من العاملين ولولا عملية الكلورة لانتشرت أوبئة الكوليرا والتيفود والتى كان لظهور الكلور واستخدامه الفضل الأكبر فى القضاء عليهما .

ويتميز التطهير بواسطة الكلور بسهولة استعماله, وكذلك سهولة الحكم على مدي فاعليته بالتأكد من بقاء قدرا من الكلور في الماء بعد فترة من إضافته .

ولا يعرف علي وجه التحديد كيف يعمل الكلورين علي قتل البكتريا في الماء وإن كان المعتقد أن مركبات الكلور التي تتكون عندما يضاف الكلور ين إلى الماء تقضى على بعض الأنزيمات الضرورية للعمليات الحيوية فى هذه الكائنات و لقد تاكد عدم صحة الاعتقاد السائد بان البكتريا تتاكسد نتيجة اضافة الكلورين وذلك لان عملية التأكسد تحتاج الى كميات هائلة من الكلور، في حين أن التطهير الناتج الآن لا يتطلب إلا كميات ضئيلة منه. وهذا لا يمنع أن الكلو رين عابل مؤكسد قوي وعندما يستخدم كميات كافية فإنه يوقف نمو الطحالب فى المرشحات .

والكلور عنصر كيماوي ينتج خلال صناعة الصودا الكاوية ويدخل الكلور في كثير من الصناعات المدنية والحربية منها علي سبيل المثال صناعة البترول والمعادن والورق والمنسوجات والصناعات الكيميائية والصناعات الغذائية كما يستعمل الكلور في معالجة مياه الصرف الصحى وتطهير مياه الثرب .

## صناعة الكلور

يتم تحضير الكلور في الصناعة بواسطة التحليل الكهربي لمحلول كلوريد الصوديوم (ملح الطعام). ويتجمع الغاز عند القطب الموجب ويتم سحبة حيث تتم إسالته ويحفظ في خزانات كبيرة إلى أن تتم عملية تعبئته في الاسطوانات المستخدمه في محطات المياه ومصانع النسيج و خلافه .

## خصائص الكلور

يمكن أن يتواجد الكلور مثله مثل أى مادة فى ثلاث صور مختلفة: السائلة، الغازية. الصلبة ولكل منها خصائصه .

## غاز الكلور

غاز لونه أصفر مائل إلى الخضرة وهو أثقل من الهواء مرتين و نصف له رائحة مميزة شديدة النفاذية ووزنه الذرى ٣٥.٥

الكلور الغازي ضعيف القابلية للذوبان في الماء (٦.٧ جم/لتر ماء عند ٢٠مْ ,١٤.٨ جم/لتر ماء عند صفر مْ ولذلك لا ينبغي رش الماء على الكلور المتسرب , كما أن حفظ محلول الكلور في درجة حرارة باردة يحافظ على . تركيزه لفترة طويلة .

وبرغم أن الكلور ليس مادة ملتهبة قابلة للاشتعال أو الانفجار, إلا انه يمكن أن يساعد على الاشتعال تحت ظروف معينة .

كذلك فإن الكلور الجاف ليس مادة مسببة للتأكل , إلا انه يتحول إلى ذلك بشكل مؤثر إذا ما تعرض للرطوبة.

غاز الكلور سام ومهيج للاغشية المخاطية المبطنة للانف والعين والجلد والرئتين ويسبب سعال وصعوبة فى التنفس.

يسال بالتبريد عند -٣٤.١ م ْ، ضغط جوى ا بار( أو يسال تحت ضغط عالي )٧كجم/ سم ٢ ، في درجات الحرارة العادية( ولذلك يحفظ وينقل علي هيئة غاز مسال بالضغط في اسصوانات من الصلب تختلف سعتها من خمسين الى ١٠٠٠ كيلوجوام وتتوقف العبوة المستعملة علي الكمية المستهلكة في محطة المياه.

فى حالة وجود تسرب لغاز الكلور فى الجو فإن ٣ جزء فى المليون هى اقل نسبة يمكن حسها بالشم ء وعند ١٥ جزء في المليون يصبح تأثيرها مهيج للعين والرئة, ويصبح خطرا إذا ما استشق لفترة من ٣٠ – ٦٠ دقيقه عند تركيز من٤٠ – ٦٠ جزء فى المليون, وإذا زاد التركيز فى الجو ليصل إلى ١٠٠٠ جزء في المليون فإنه يصبح مميت. أما الكمية المسموح بها لغاز الكلور في الجو وتكون آمنة لفترة الوردية الواحدة (ثماني ساعات) لا تتجاوز تركيز ١ . ٠ جزء في المليون.

## الكلور السائل

عبارة عن محلول نقى كهرماني اللون وهو أثقل من الماء مرة ونصف تقريبا. وللكلور السائل معامل تمد د عالي، إذ يزداد حجمه بسرعة كبيرة بازدياد درجة الحرارة, حيث يزداد تمدد السائل ليملاء الأسطوانة بالكامل عندما ترتفع درجة الحرارة إلى ٦٧.٥ مْ ولذلك يلتزم دائما بعدم ملء اسطوانات الكلور بأكثر من ٨٥ % من حجمها. وحيث أن الكلور السائل يتبخر بسرعة شديدة إذا ما تعرض للهواء الجوى, لذلك فهو نادرا ما يرى في صورته السائلة.

وعند تبخر الكلور السائل فإن وحدة الحجوم الواحدة منه تنتج حوالي ٤٥٦ وحدة حجم من الغاز النقى عند درجة حرارة ١٥ مْ وضغط ٠ ٧١ مم زئبق و بالتالى عند حدوث تسرب فى اسطوانة الكلور يتحتم تعديل وضع الاسطوانة بحيث تكون منطقة التسرب فى اعلاه لكى يتسرب غاز الكلور و ليس السائل

## الكلور الصلب

نظرا لان الكور السائل يتجمد عند درجة حرارة منخفضة جدا ( -١٠٢ مْ ) فهو نادرا ما يوجد في صورته الصلبة , غير انه يتواجد متحدا مع بعض العناصر الأخرى فى صورة مركبات على هيئة بودرة أو حبيبات .

## مركبات الكلور

بخلاف الكلور النقى يتواجد الكلور علي هيئة مركبات سائلة أو صلبة :

- **محلول الكلور**

ويسمي كيماويا (هيبوكلوريت الصوديوم وهو محلول يحتوي علي حوالي ١٥% من الكلور الحر ويباع في المحلات العامة كمنظف تحت أسماء مختلفة, ويمكن استعماله كمطهر للمياه بشرط أن لا يضاف إليه أي أنواع من المنظفات ويضاف مباشرة على الماء سواء بالصب المباشر من وعاء أو باستعمال مضخة مناسبة. (لا يستعمل بكثرة بالإضافة إلى آن محلوله يسبب تآكك في المواسير) .

- **مسحوق أو أقراص الكلور:**

ويسمي كيماوياً (هيبوكلوريت الكا لسيوم) و هذه المادة تحتوى علي حوالي ٦٠ – ٧٠ % من الكلور الحر. ويمتاز عن المسحوق المبيض بارتفاع نسبة الكلور الفعال وبان نسبة الكلور الفعال لا تتأثر بالتخزين لفترات ت ليست بالطويلة. وعند الاستعمال يحضر محلول مركز منه ثم يضاف إلى الماء بالجرعات اللازمه بواسطة أجهزة خاصة .

- **المسحوق المبيض:**

ويسمي أحيانا (كلوريد الجير أو الجير المكلور وهو مسحوق ابيض مائل للاصفرار, له رائحة قوية نفاذة، الجديد منه يحتوي علي ٣٢ % من وزنة كلور فعال. إلا أن هذه النسبة تأخذ فى النقصان بمضي الوقت خصوصا إذا تعرض للجو أو للضوء، ولذلك يجب حفظة في عبوات خاصة محكمة القفل , كما يجب اختباره لمعرفة نسبة الكلور الفعال قبل الاستخدام حتى يمكن تقدير الكمية التي تعطي جرعة الكلور المطلوبة.

وعموما فإن اسعمال مركبات الكلور اصبح غير شائع فى عمليات التطهيرللمياه نظرا لمتاعب التشغيل إلا انه يستعمل فى الحالات الآتية:-

١ . تطهير شبكات مواسير توزيع المياه بعد إنشائها او إصلاحها.

٢ . تطهير مرشحات وخزانات المياه

٣. في حالات الطورئ مثل حالات الفيضانات.

## استخدامات الكلور فى محطات المياه

يستخدم الكلور فى محطات المياه لغرض التطهير و للقضاء على مشاكل الطعم و الرائحة و لاكسدة الشوائب الكيميائية

**١. التطهير:**

يستخدم الكلور فى محطات مياه الشرب كمادة مطهرة. إذ أن إضافة اكلور تتسبب فى قتل البكتريا الضارة والمسببة للامراض. ولذلك تحدد اشتراطات جهات الصحة المختلفة ضرورة

إضافة جرعات وافية من الكلور للماء تكفل توافر الكلور المتبقى حت ى نهايات شبكة التوزيع ( فى حدود ٢ . ٠ ملجم/لتر)

**٢. القضاء على مشاكل الطعم والرائحة:**

قد يستخدم الكلور للقضاء على بعض مشكلات الطعم والرائحة فى مياه الشرب. إذ أن معظم مشاكل الطعم والرائحة تتسبب عن بعض النباتات المجهرية الدقيقة التى تتواجد فى مصادر المياه السطحية. فيتفاعل الكلور كمادة مبيده للطحالب ويقضى على تلك النباتات المسببة للطعم والرائحة. إلا أن هناك أنواع قليلة من الطحالب لا تتفاعل مع الكلور بالشكل المرغوب ويترتب على ذلك زيادة حدة الروائح الطعم غير المقبول للماء.

**٣. الأكسدة:**

يقوم الكلور بأكسدة عدد من الشوائب الكيميائية الموجودة فى الماء كالحديد. المنجنيز , النيترات , الأمونيا وكبريتيد الهدروجين . وبالتالى ازالتها من الماء. وتتواجد تلك الشوائب عادة فى مصادر المياه الجوفية .

## استخدامات الكلور فى معالجة مياه الصرف الصحى

يستخدم الكلور فى معالجة مياه الصرفالصحى لأغراض التطهير والتحكم فى الرائحة والمساعدة على الترسيب وتخفيض الأكسجين الحيوى الممتص وإ زالة الخبث الطافى تكتلات الحمأة والقضاء على ظاهرة تكون البرك وانتشار الذباب فى المرشحات الزلطية .

**١. التطهير:**

إن المياه المعالجة والتى يتم صرفها فى المجارى المائية , لابد وأن تخضع أولاً لعملية الكلورة. ويتم ذلك بإضافة الجرعات الكافية من الكلور والتى تكفل بقاء١ ملجم/لتر على الأقل من الكلور المتبقى فى الماء بعد فترة تلامس ٢٠ دقيقة. وذلك لضمان القضاء على المواد العضوية الضارة والمسببة للامراض قبل صرفها على المجرى المائي.

**٢. التحكم فى الرائحة:**

ترجع الروائح الموجودة فى محطات معالجة مخلفات الصرف عادة إلى التحلل اللاهوائى لمخلفات المواد العضوية. فإذا كانت المخلفات الداخلة لمحطة المعالجة فى حالة تعفن أو تحلل، استلزم ذلك إضافة الكلور إلى التصرف الداخل.

أما المخلفات ذات معدلات الأكسجين الممتص العالية كمخلفات صناعات التعبئة والتغليف, ومعامل تعليب الأطعمة و ومزارع الأ لبان ء والمخلفات الصناعية والتجارية الأخرى. فيجب إضافة الكلور إليها قبل وصولها إلى مجارى الصرف. ويعد توفير ٥٠ % من استهلاك الكلور معدلاً جيداً للقضاء على الروائح الكريهة .

**٣. المساعدة على الترسيب:**

فى الأحوال التى تظهر فيها بوادر التحلل أو التعفن في أحواض الترسيب الابتدائي، تفيد عملية كلورة المخلفات الأولية فى تحسين _قدرة المواد الصلبة العالقة على الترسيب. إذ أن المواد الصلة المتحللة تنتج غازات تؤدى إلى زيادة المواد الصلبة العالقة فى المروق.

**٤. تخفيض الأكسجين الحيوى الممتص :-**

يعد الأكسجين الحيوى الممتص مقياساً لقوة تركيز المخلفات أو لمقدار المواد العضويه الغير مسقره الموجوده بها. ويقوم الكلور بخفض معدل الأكسجين الحيوى الممتص أو بتقيلل تركيز مخلفات الصرف السائلة عن طريق أكسدة بعض المواد العضوية الغير ثابته. وبإضافه١ ملجم/لتر كلور ممكن توقع خفضمعدلالاكسجين الحيوى الممص بمقدار ٢ ملجم/لتر عند الدرجة التى يتوفر فيها الكلور المتبقى.

**٥. ازالة الشحم**

يمكن استخدام الكلور فى استحلاب الشحوم( تحويلها إلى مستحلب) مما يسمح بكشطها من السطح بعد ذلك. كما يتم استخدام الهواء أحياناً بإدخاله من خلال قاع الحوض/الخزان ليساعد على طفو الشحم المستحلب

### ٦. ازالة الخبث الطافى و تكتلات الحماءة

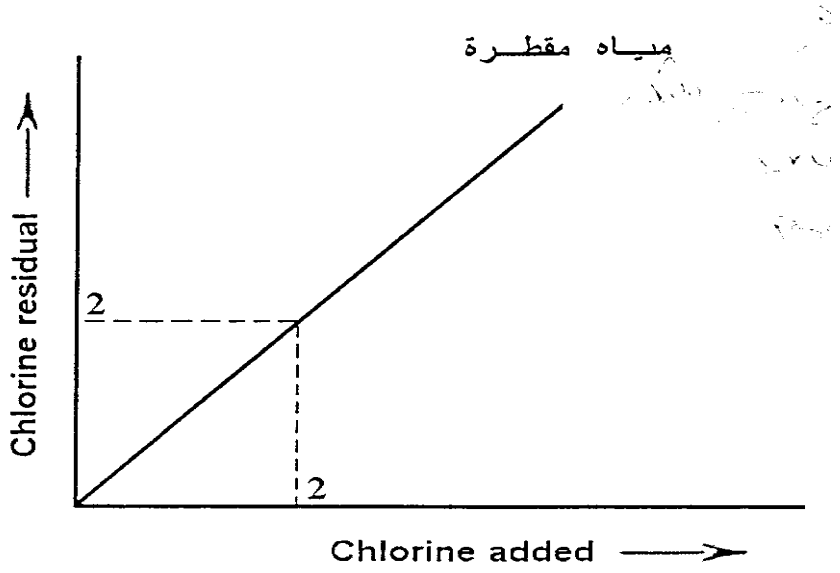
يستخدم الكلور فى بعض الأحيان للتحكم فى الحمأة الطافية داخل محطة الحمأة المنشطة عن طريق كلورتها. كما يمكن أيضا استخدام الكلور فى محطات الحمأة المنشطة للمعادلة فى حالات التحميل الزائد .

### ٧. القضاء على ظاهرة تكون البرك وانتشار الذباب فى المرشحات الزلطية

قد يستخدم هيبوكلوريت الكالسيوم فى المرشحات الزلطية التى تظهر بها مشكلة تكون البرك (انسداد وسط المرشح وركود المياه ) وتنتج هذه الظاهرة عادة عن وجود كثافات كبيرة جداً من المواد الهلامية (كتل هلامية من البكتريا تتشكل عند انتفاخ جدران الخلية نتيجة لامتصاص الماء). ويؤدى استخدام الكلور إلى قتل هذا النمو الزائد والذى ينسلخ من تلقاء نفسه بعد ذلك وينجرف الى المروق النهائى. هذا بالإضافة إلى أن استخدام الكلور يساعد أيضاً فى القضاء على يرقات الذباب بالمرشح.

# نظرية عمل الكلور

لفهم تفاعلات الكلور في المياه الطبيعية . نفرض مبدئياً أن التفاعل سيحدث في مياه مقطرة . فإن كمية الكلور الحر المتبقية تتعلق مباشرة بكمية الكلور المضافة( الجرعة .)




وعلى سبيل المثال: إذا اضيف جرعة كلور بمقدار ٢ مجم/لتر لتلك المياه ستعطى نفس القيمة ( ٢ مجم/لتر) ككلور متبقى حر بعد انتهاء فترة المكث .

**وسيتم التفاعل كالتالى:**

كلور + ماء ← حامض الهيبوكلورز + حامض الهيدروكلوريك

$CL_2 + H_2O \rightarrow HOCl + HCl$ ….. (1)

**نواتج التفاعل مركبات ضعيفة لا تلبث ان تتفكك إلى نواتج أخرى كالتالي:**

حامض الهيبوكلورز ← أيون الهيبوكلوريت + ايون هيدروجين

(حامض ضعيف يتفكك)

$HOCl \rightarrow OCl^- + H^+$ ….. (2)

وحامض الهيدروكلوريك ← ايون كلور + ايون هيدروجين

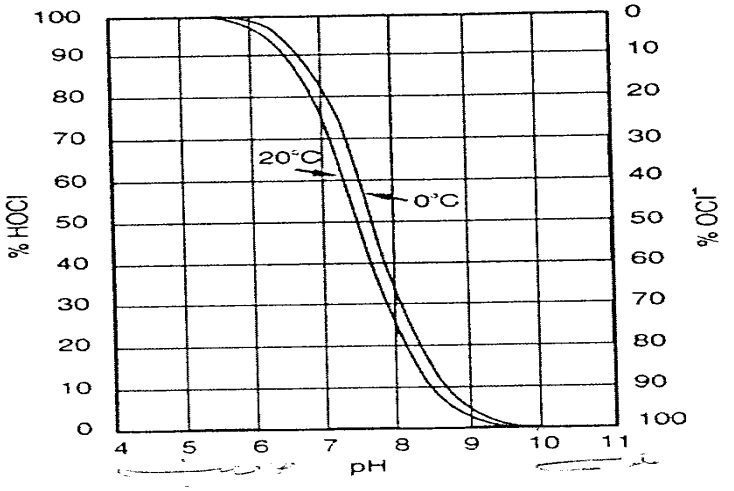
$HCl \rightarrow Cl^- + H^+$ ….. (3)

ويمثل حمض الهيبوكلورز HOCl أحد شكلين من أشكال الكلور الحر المتبقى وهو الأكثر فاعلية في عملية التطهر وعند تحلله كما في المعادله ( ٢ ) السابقه ينتج ايون الهيبوكلورت-OCl وهو الشكل الثانى من أشكال الكلور الحر التبقي وتأثيره في عمله التطهير تعادل ١% فقط من تأثتر حمض الهيبوكلورز.

| الشكل | فاعلية الكلور مقارنة بحمض الهيبوكلورز |
|---|---|
| HOCl | 1 |
| OCl – | 1/100 |
| NCl3 | - |
| NHCl2 | 1/80 |
| NH2CL | 1/150 |

## تأثير pH علي التفاعل:

يتأثر التفاعل بالأس الهيدروجيني تأثيرا كبيرا فكلما كان الوسط اقرب للحامضية زاد حامض الهيبوكلوروز وكلما كان الوسط اقرب للقاعدية زادت أيونات الهيبوكلوريت.



## فمثلا :-

**عند الاس الهيدروجينى ٦.٧ تكون النسبة**

٢٠ % ملح هيبوكلوريت +٠ ٨ % حمض هيبوكلوروز

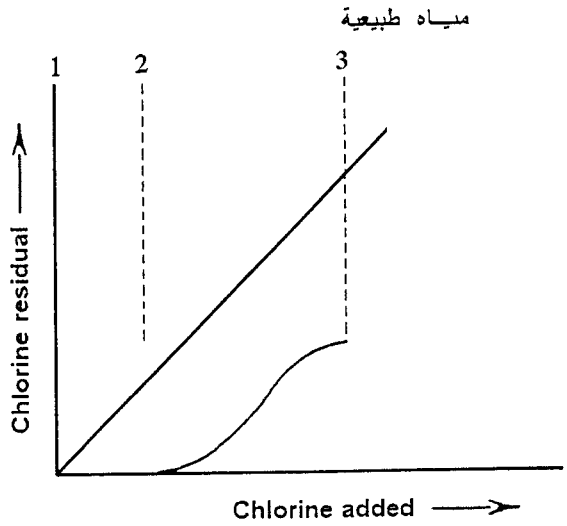
**عند الاس الهيدروجينى ٧.٣ تكون النسبة**

٤٠ % ملح هيبوكلوريت +٠ ٦ % حمض هيبوكلوروز

**عند الاس الهيدروجينى ٨.٣ تكون النسبة**

٨٠ % ملح هيبوكلوريت + ٢٠ % حمض هيبوكلوروز

يتبين مما سبق انه قد يوجد كلور بالعينة. ولكن فاعليته تختلف بالزيادة او النقصان تبعا لاختلاف درجة تركيز الأس الهيدروجينى




أما بالنسبة للمياه الطبيعية فكما نعلم أن بها مواد ذائبة وشوائب عالقة وبالتالى فان تفاعل الكلور مع تلك المواد والشوائب سيتداخل ويؤثر على كمية الكلور المتبقى الحر كنتيجة لذلك.

**وعلى سبيل المثال:** إذا احتوى الماء على حديد، منجنيز, أمونيا, نيتريت , أو أي مادة عضوية فسوف يتفاعل الكلور المضاف معها ليعطى انحراف في المنحنى يناسب تلك المتفاعلات ودرجة تركيزها بالمياه

من الشكل:- المسافة بين النقطتين ٢ و ١ الكلور المضاف يتحد مباشرة مع الحديد و المنجنيز و النيتريت كمواد مختزلة ولا يظهر تبعا لذلك أي كلور متبتقى حتى الانتهاء من أكسدة كل تلك المواد بواسطه الكلور.

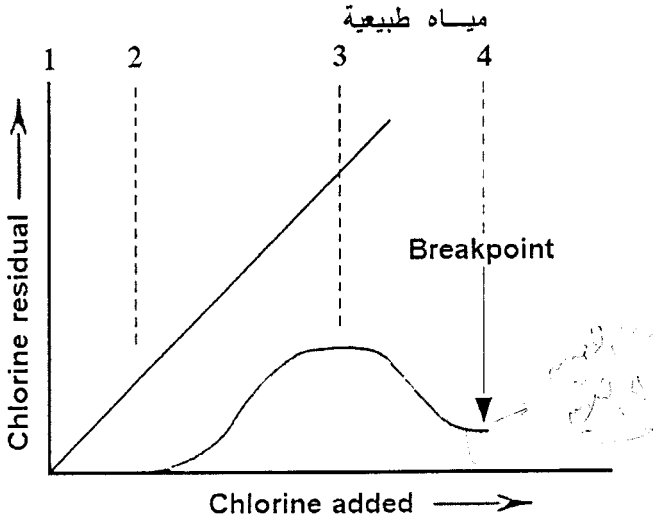
باستمرار إضافة الكلور نلاحظ انه على امتداد المسافة بين النقصتن ٣ ، ٢ يبدأ الكلور في التفاعل مع الأمونيا والمادة العضويه ليشكل مركب الكلورامين والمركبات الكلو وعضوية وهذه النواتج من أشكال الكلور تعرف بالكلور المتبتقى المتحد ولكن فاعلة هذه الأ شكال كعامل مطهر تكون اضعف من الكلور الحر.

وفيما بين النقطتين ٣ و ٢ غالبا يكون أحادى الكلورامين هو شكل الكلور المتبقى كناتج التفاعل:

أمونيا + حمض الهيبوكلورز ← أحادى الكلورامين + ماء

$$NH_3 + HOCl \rightarrow NH_2Cl + H_2O \quad (pH > 7) \ldots\ldots (4)$$

يوضح الشكل التالى انه بمتابعة إضافة كلور للمياه , يبدأ الكلور المتبقى من النقطة ٣ حتى النقطة ٤ في الانخفاض وذلك كنيجة لبدأ أكسدة مركبات أحادى الكلورامين الى مركبات ثاني وثالث الكلورامين و حتى تمام الأكسدة وتكون غاز النيتروجين .



ويوضح ذلك التفاعلات التالية:

في حالة إذا ما كان $NH_3$ بكمية وفيرة:

$$NH_2Cl + HOCl \rightarrow NHCl_2 + H_2O \quad (4 < pH < 6)..... (5)$$

$$NHCl_2 + HOCl \rightarrow NCl_3 + H_2O \quad (pH < 3)..... \quad (6)$$

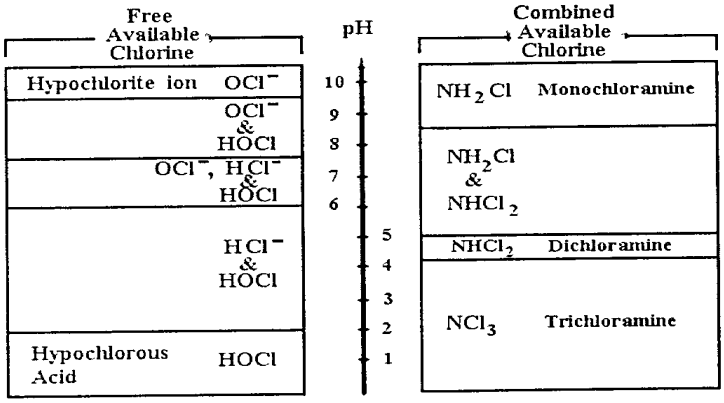
ونلاحظ من التفاعلات السابقة انه عند pH اكبر من ٧ يتكون أحادى الكلورامين , وعند pH 6 يبدأ ثاني الكلورامين ببطئ في التكوين ويظهر بوضوح بين ٦ <pH>٧ أما ثالث الكلورامين فيتكون عند pH أقل من ٣ . كما أن الكلورامين ممكن أن يتك,ن مع أي Amine Organic منل البروتين، حيث يظهر الكلورامين بسرعة أولا ثم يعقبه ببطء ظهور ثاني وثالث الكلورامين مع الاحتياج إلى فترة تلامس أطول حتى يتم التفاعل.

إذا كان كمية $NH_3$ في الماء كافية فإن التفاعل سيكون ثابت , وإذا ما كان كمية الكلور المضافة اكبر مع $NH_3$ فان الكلورامين سيتكسر إلى النير وجين

كما في المعادلة التالية : -

$$2NH_2Cl + HOCl \rightarrow N_2 + 3\,HCl + H_2O \ldots (7)$$

٦-) يوضح علاقة pH بأشكال الكلـــور المحتمـــل تواجـــدها بالميـــاه

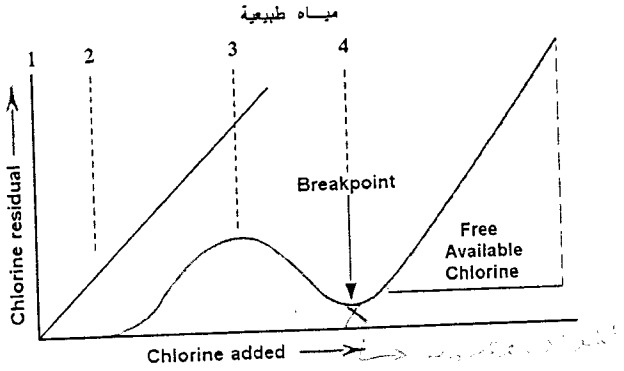



ويختلف مفعول كل نوع من مركبات الكلور السابقة عن الآخر في عمليات التطهير حيث يلاحظ أن الكلور الحر اقوي بكثير في قتل البكتريا عن الكلور المتحد. ويلا حظ أيضا أنه يلزم استخدام

مقدار من الكلور المتحد اكبر ٢٥ مرة من الكلور الحر لتحقيق نفس القدرة علي القتل في نفس المدة الزمنية

**علاوة علي ذلك فان التطهير بالكلور من عملية بطيئة وتحتاج إلى فترة تلامس اطول.**

النقطة ٤ هى أدنى قيمة يصل لها الكلور وتعرف باسم نقطة الانكسار. ثم يبدأ بعدها ظهور كلور حر متبقي ويزيد بزيادة جرعة الكلور المضافة . بمعنى انه بعد نقطة الانكسار أي إضافة للكلور تظهر في صورة كلور حر متبقي, وقيمة الكلور الحر المتبقى يجب أن تمثل ٨٥ –٩٠ % من مجموع الكلور المتبقى (حر + متحد) بالمياه المعالجة .




يجب أن نتذكر أن الكلور المتبقى المتحد يمثل مركات ثنائى و ثلاثى الكلورامين والمركبات الكورو– عضوية )As THMs

## العوامل المؤثره فى عملية التطهير ( فاعلية الكلور )

**تتوقف فاعلية الكلور في قتل البكتريا علي العوامل الآتية:–**

**١. جرعة الكلور:**

بديهي إن فاعلية الكلور فى القضاء علي البكتريا تزيد بازدياد جرعة الكلور المضافة الى المياه.

**٢. مدة التفاعل بين الكلور والماء ( فترة التلامس)**

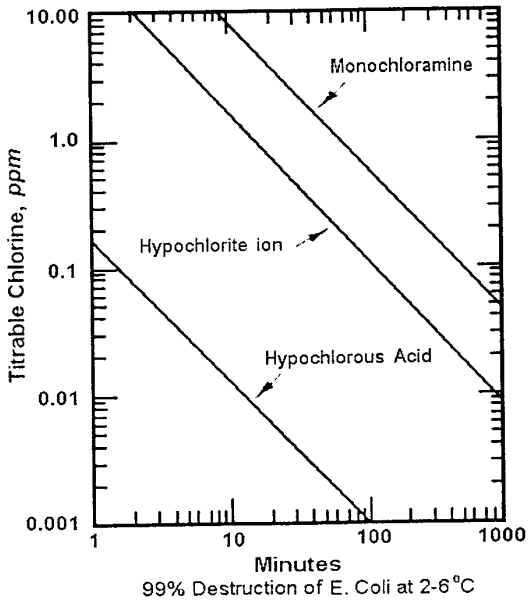
تزيد فاعلية الكلور كلما طال من التلامس . و نظرا لان مقاومة البكتريا للكلور باختلاف أنواعها . فوجد انه يجب أن تمر ثلاثين دقيقة على الأقل من إضافة الكلور وحتى استعمال الماء كفترة تلامس. وقد وجد أن قدرة الكلور على قتل البكتريا بالماء تتأثر تأثراً كبيراً بالعاملين السابقين (الجرعة، ووقت التلامس ) طبقا للعلاقة التالية:

$$\text{Kill } \alpha \text{ CXT}$$

ويؤثر كل منهما على الآخر فبزيادة تركيز الكلور يقل الوقت اللازم للقتل والعكس بالعكس.مع الأخذ في الاعتبار أن الكلور المتحد تأثيره اقل من الحر في عملية التطهير ولذلك يلزمه فترة تلامس أطول لإتمام عملية قتل البكتريا الموجودة بالماء.

من المهم معرفة فترة التلامس الملائمة لنوع الكلور المتبقى الموجود بالماء , فمثلا نعتمد على الكلور الحر المتبقى في التطهير عندما يكون الوقت المقطوع من نقطة إضافة الكلور حتى نقطة استهلاكه قصير. الشكل ( يبين العلاقة بين الوقت اللازم لقتل بكتريا E Coli بنسبة ٩٩ % عند درجة حرارة ٢ – ٦ م˚ ونوع الكلور المتبقى بالماء .

**تذكر انه :** يجب أن يتوفر ٢ . ٠ مجم/لتر كلور حر فى أطراف الشبكة التى تتغذى من محطة المعالجة.



**٣. درجه الحراره:**

بالرغم من أن درجة حرارة المياه المنخفضة تحتفظ أكثر بالكلور إلا أن الحرارة العالية أكثر فاعلية، ولذلك يجب تغيير جرعة الكلور مع تغير الفصول للحفاظ على نسبة الكلور المتبقى بالمياه, بمعنى إن جرعة الكلور تزيد بانخفاض درجة الحرارة للحصول علي نفس كفاءة التطهير.

**٤. درجة تركيز أيون الهيدروجين:**

لقد تبين أن المياه ذات التركيز الهيدروجيني المنخفض تلزمها جرعات كلور اصغر من المياه ذات التركيزالهيدروجين المرتفع للحصول علي نفس كفاءة التطهير.وتفسير ذلك ان الأس الهيدروجيني هو الذي يحدد النسبة بين تركيزي كل من -OCl - HOCl ويوضح الشكل السابق تلك العلاقة. لاحظ من الشكل أيضاً أن درجة الحرارة تأثيرها ضعيف على نسبة التحلل بالمقارنة بتأثير الأس الهيدروجيني.

استخدام مركبات الهيبوكلوريت في عمليات التطهير ترفع من قيمة pH قليلا أما استخدام غاز الكلور فيخفض من pH ، ولذلك من الأهمية متابعة القياس والتحكم في درجة الأس الهيدروجيني للمياه للمحافظة على درجة تأثير الكلور في عملية التطهير .

**٥. عكارة الماء:**

كلما زادت عكارة الماء زادت جرعة الكلور اللازمة إذ أن الميكروبات قد تحتمي بالمواد المسببة للعكارة من تأثير الكلور.

**٦. قلوية وحامضية الماء:**

وتقل فاعلية الكلور بزيادة قلوية الماء ولذلك يلزم استخدام جرعات كلور عالية كلما ارتفعت قلوية الماء.

**٧. وجود المركبات الآزوتية في الماء:**

عند تواجد مركبات الأمونيا في المياه تقل فاعلية الكلور في قتل الكائنات الحية في الماء ولذا يلزم إضافة جرعات اكبر أو إطالة وقت التفاعل بين الكلور والماء ( فترة التلامس )

**٨. وجود مركبات الحديد والمنجنيز:**

وهذه أيضا تحد من فاعلية الكلور في قتل البكتريا.

# اضافة الكلور

يتم إضافة الكلور للمياه بإحدى طريقتين:

- إضافة الكلور بجرعات عالية ثم إزالة الكلور الراند.
- إضافة الكلور بعد تحديد النسبة بواسطة التجارب المعملية الدقيقة.

**أولأ: إضاقة الكلور بجرعات عالية ثم إزالة الكلور الزائد :-**

والمقصود بذلك إضافة الكلور بجرعات زائدة عن حد الطلب, وبهذا نضمن الحصول على كفاءة وفاعلية عالية لعملية التطهير وممكن أن نضطر إلى ذلك فى حاله الطوارئ أو إذا ما كان هناك شك في حدوث تلوث بكتيري لمصدر المياه.

**و تتميز هذه الطريقة بالاتى :-**

- كفاءة وفاعلية عالية لتأثير الكلور علي البكتريا.
- أكسدة الكلور للمواد العضوية التي قد تتواجد في الماء.
- الحد من الطعم والرائحة التي قد توجد في الماء.
- إبادة الكائنات الحية الدقيقة التي تقاوم الجرعات العادية للكلور، علي انه يلزم إزالة الكلور الزائد بعد التأكد من تمام قتل الكلور للبكتريا.

**طرق إزالة الكلور الزائد :**

١ -إضافة ثاني أكسيد الكبريت إلى الماء بجرعات حوالي ١.٥ جزء فى المليون لكل جزء في المليون من الكلور المراد إزالته وفي هذه الحالة يتفاعل ثاني أكسيد الكبريت مع الكلور الزائد كما هو موضح بالمعادلة:

$$SO_2 + H_2O + Cl_2 \rightarrow 2HCl + H_2SO_4$$

كميات حامض الكبريتيك وحامض الهيدروكوريك الناتجة من التفاعل ضئيلة جدا ولا أهمية لها كما يجب أن تمر حوالي ١٥ دقيقة بعد إضافة ثانى اكسيد الكبريت قبل استعمال المياه.

٢ -إضافة ثيوسلفات الصو.يوم إلى الماء ليتفاعل مع الكلور الزائد كما هو موضح بالمعادلة :

$$2\ Na_2S_2O_3 + Cl_2 \rightarrow Na_2S_4O_6 + 2\ NaCl$$

٣ -إضافة كبريتيت الصوديوم إلى الماء ليتفاعل مع الكلور الزائد كما هو موضح بالمعادلة:

$$Na_2SO_3 + Cl_2 + H_2O \rightarrow Na_2SO_4 + 2\ HCl$$

٤ -تخزين الماء في أحواض مكشونة لمدة ثلاث أو أربع ساعات قبل الا ستعمال وفي هذه الفترة يتصاعد الكلور الزائد فى الجو .

٥ -ترشيح الماء فى طبقة من الكربون المنشط الذى يمتص الكلور الزائد

٦ -مزج المياه المضاف إليها جرعات عالية من الكلور بمياه لم يضاف إليها الكلور فتتعادلان .

**ثانياً: إضافة الكلور بعد تحديد النسبة بواسطة التجارب المعملية**

تحديد احتياج الكلور باختبار Break point

احتياج الكلور هو مقياس لمقدار الكلور الواجب إضافته لوحدة الحجوم من المياه تحت شروط محدودة للاس الهيدروجينى ودرجة الحرارة وفترة التلامس حتي يتم التفاعل الكامل بجميع مواد

الكلور القابلة للتفاعل فى الماء ويحقق كلور حر متبقي، ويحسب احتياج الكلور بأنه الفرق بين كمية الكلور المستخدم والكلور المتبقى في المياه عند فترة التلامس

## المصطلحات الفنية الخاصة بالكلور

### جرعة الكلور

تعرف جرعة الكلور بأنها اقل كمية كلور تضاف إلى وحدة حجم من الماء تكفى للقضاء على اللكائنات الحية، وينتج عنها كلور متبتقى في حدود معينة (٠.٢ مجم/لتر)

ووحدة الجرعة ملجم كلور/لترماء أو جم كلور/ م$^3$ماء.

و يتم تحديد الجرعة المثلى للكلور عن طريق تجارب معملية حسب نوعية المياه المراد تنقيتها

**كمية الكلور المطلوب اضافتها للماء:** هى حاصل ضرب جرعة الكلور X كميه المياه وهى غالباً تحدد فى الساعة ووحدتها كجم/ساعة

**مثال:**

ما هى كمية الكلور المطلوب إضافتها تكفى لتطهير ٢٠٠٠ م $^3$ / س بجرعة ٥ جم/ م$^3$

$$\text{كمية الكلور المطلوبة} = \frac{٢٠٠٠٠\text{م}^3/\text{س} \times ٥\text{جم}/\text{م}^3}{١٠٠٠} = ١٠٠\text{كجم}/\text{س}$$

وفى حالة إضافة هيبوكلوريت الكالسيوم تركيز ٦٥% بدلاً من الكلور دون حدوث أى تأثير على المياه، فإنه يلزم أولاً معرفة كمية الكلور المطلوب إضافتها.

كمية الهيبوكلوريت التى تعوض كمية الكلور =

$$\frac{\text{كمية الكلور} \times ١٠٠}{\text{نسبة تركيز الهيبوكلوريت}}$$

**مثـــال:**

ما هى كمية هيبوكلوريت الكالسيوم (٦٥%) التى تضاف إلى المياه بدلاً من ١٠٠ كجم كلور.

$$\text{كمية هيبوكلوريت الكالسيوم (٦٥\%)} = \frac{١٠٠ \times ١٠٠}{٦٥} = ١٥٣,٨٥ \text{ كجم}$$

## الكلور المستهلك

يعرف الكلور المستهلك بأنه الفرق بين كمية الكلور المضاف للماء وكمية الكلور الحر أو المتحد المتبقى في الماء فى فترة تلامس محددة

الكلور المستهلك = كمية الكلور المضاف – كمية الكلور المتبقى

**مثال:**

ما هى كمية الكلور المستهلك إذا كان الكلور المتبقى ٢ . ٠ جم/ م$^٣$ بعد إضافة ١٠٠ كجم كلور لكمية مياه مقداره ٢٠٠٠٠ م$^٣$ ؟

**الحل**

الكلور المستهلك = كمية الكلور المضاف – كمية الكلور المتبقى

كمية الكلور المتبقى =٢ . ٠ جم/ م$^٣$ X ٢٠٠٠٠ م$^٣$ = ٤٠٠٠ جم = ٤ كجم

اذا الكلور المستهلك = ١٠٠ – ٤ = ٩٦ كجم

هذه الكمية استهلكت بواسطة الكائنات الحية والمواد العضوية وغير العضوية والشوائب.. إلخ. ولابد من وجود كلور متبقى بعد كل هذه الاستهلاكات. وهذا دليل على أنه لم يعد هناك أى كائنات حية أو خلافه وأن الماء اصبح خاليا منها و معقم تماما

## الكلور المتبقى

هناك نوعان من الكلور المتبقى

**١. الكلور المتبقى المتحد:**

ينتج عن إضافة قدر من الكلور يكفى فقط للاتحاد مع الامونيا الموجودة بالماء . وبرغم أن تلك البقايا المتحدة تحمل قدرات أكسده تفوق قدرة الكلور الحر إلا أن فعاليتها كمادة مطهرة تقل عن فاعلية الكلور الحر. وللحصول على الكلور المتبقى المتحد لا يضاف إلى الماء إلا قدر الكلور الذى يكفى للاتحاد مع الأمونيا الموجودة فى الماء , فإذا كان الماء يحتوى على قدر ضئيل من الأمونيا لا يكفى للاتحاد مع الكلور فإنه يتم إضافة الأمونيا أيضاً إلى. الماء .

**٢. الكلورالمتبقى الحر:**

ينتج عن إضافة الكلور إلى الماء بالقدر الذى يكفى لاستهلاك الأمونيا الموجودة فى الماء والقضاء عليها. والكلور المتبقى الحر كما ذكرنا قبل هذا أكثر فعالية من الكلور المتبقى المتحد كمادة مطهرة.

## اماكن إضافة الكلور في محطات التنقية

يمكن إضافة الكلور إلى الماء في أكثر من موقع في محطات التنقية تبعا لحالة كل محطة وكذلك تبعا لصفات الماء المعالج في كل حالة وتبعا لتجارب وخبرات المشرف علي التشغيل.

**١.اضافة الكلور المبدئى**

اى حقن الكلور بعد عملية الحصول على المياه من المصد ر مباشرة وقبل الدخول الى عملية التنقية (المروقات والمرشحات )

**تتميز هذه الطريقة بالاتى :-**

- خفض تعداد. البكتريا في الماء قبل وصولها إلى المرشح مما يخفف الحمل البكتيرى على المرشح.
- تطهير رمل المرشح نظرا لمرور المياه بما فيها من كلور فى مسام الرمل أثناء عمله الترشح.
- كفاءه عاليه في إزاله اللون من الماء.
- نقص في كميات الكيماويات المروبه إذا أضيف الكلور قبل أحواض الترسيب.
- كفاءة عايله في إزاله الطعم والرائحه من الماء.
- الحد من نمو الكائنات الحيه الدقيقة داخل المرشح.

**٢.اضافة الكلور النهائى**

أي إضافة الكلور إلى الماء بعد مرحلة الترشيح أي عند مدخل خزان المياه النقية , وهي طريقة سهلة في تشغيلها ويكون الكلور أكثر فاعلية علي البكتريا بسبب خلو الماء من أي عكارة أو شوائب.

يضاف جرعة زائدة من الكلور النهائي في الخزانات الملحقة بالمحطات وذلك لضمان خروج الكلور بنسبة معينة مطلوبة إذا كانت شبكات مواسير التوزيع تمتد الى مسافات بعيدة ويخشى من تواجد البكتريا فى الأطراف البعيدة منها وحتى يصل الكلور ولو بنسبة ضئيلة إلى آخر متر في الشبكة.

**٣. .إضافة الكلور في اكثر من موقع**

وهذه الطريقة تتبع إذا كانت المياه رائقة والتلوث البكتيري عالي نسبيا إذ يحسن في هذه الحالة إضافة الكلور في أكثر من نقطة علي مسار الماء في محطة التنقية لضمان كفاءة عملية الكلوره كما تستعمل هذه الطريقة إذا خزنت المياه المرشحة في خزانات مكشو فةففى مثل هذه الحالة يجب إضافة الكلور في مخارج المياه من الخزانات .

** تذكر انه من نتائج أضافه المواد المؤكسدة (الكلور) للمياه :

١. تكسير أغلب المركبات المسببة للون .
٢. أكسدة الحديد والمنجنيز ومعظم المركبات الكبريتية

٣. تكسير بعض المركبات المسببه للرائحه .
٤. تكسير بعض المر كبات الحلقية .
٥. القضاء علي عدد كبير من البكتريا .

# الفصل التاسع

## مصادر وخصائص المياه

# الباب التاسع

# مصادر وخصائص المياه

الماء هو شريان الحياة الرئيسي حيث بدونه لا يمكن أن توجد حياه على سطح الأرض ، يغطي الماء بحالته السائلة أو الصلبة أربعة أخماس كوكب الأرض وتناقص المياه هو أحد المشكلات التي تواجه العالم في هذه القرن وذلك نتيجة الزيادة المطردة في تعداد سكان العالم الذي ترتب عليه زيادة الرقعة العمرانية وكذلك الأنشطة الصناعية التي تستهلك كميات كبيرة من المياه وينتج عنها ملوثات تغير من مواصفات مصادر المياه الأمر الذي يستلزم اتخاذ إجراءات من الحكومات لسن التشريعات ووضع المواصفات القياسية لنوعية المياه علاوة على تطبيق التكنولوجيات التي من سبيلها الحد من الاستهلاك المطرد والوصول إلى مياه نقية صالحة للاستخدام .

## الدورة الهيدرولوجية للمياه:

يختص علم الهيدرولوجيا بدراسة توزيعات المياه في الكرة الأرضية وبحركتها المستمرة من البحار إلى الجو ومن الجو إلى اليابسة ومن الأرض عودا إلى البحار وتسمى هذه الدورة بالدورة الهيدرولوجية ، ومعظم المياه التي تسقط على الأرض لا تصل إلى المحيطات بل تكمل دورتها الهيدرولوجية عودا إلى الجو بعملية البخر وعملية نتج النباتات

وحوالي ٢٥%من المياه العذبة بالكرة الأرضية مخزونة تحت سطح الأرض حيث تبقي لمئات أو آلاف السنين ، ونسبة صغيرة منها تكون موجودة في طبقات يمكن سحبها منها بكميات محددة .

وعليه يمكن تلخيص مصادر مياه الشرب كالآتي :

١-الأمطار ٢-البحار ٣-المحيطات

٤-البحيرات ٥-الأنهار ٦-المياه الجوفية

## درجة نقاء الماء

تأتي مياه الأمطار في المقدمة من حيث النقاء لاحتوائها على نسبة بسيطة من المواد العضوية علاوة على بعض الغازات الذائبة ، ثم تليها المياه الجوفية التي تحتوي على بعض أملاح الكالسيوم و الماغنسيوم والصوديوم وبعض المواد العالقة وذلك طبقا لنوعية التربة التي تمر بها .

أما المياه السطحية فتأتي في المؤخرة وهي عبارة عن مياه البحار والمحيطات والبحيرات والأنهار ، وتعتبر كلها مصادر لمياه الشرب ، فيتم الحصول على مياه الشرب من مياه البحار والمحيطات عن طريق عمليات التحلية ، أما مياه الأنهار والبحيرات العذبة فيتم الحصول على مياه الشرب منها عن طريق عمليات المعالجة ، أي أن المياه السطحية هي المصدر الرئيسي للمياه.

والمياه الجوفية عبارة عن مياه مستخرجة من الآبار ، وتكون أملاحها عادة أكثر من المياه السطحية إلا إنها تحتوي على نسبة بسيطة جدا من المواد العالقة ، لأن مرور المياه في طبقات الأرض يرشحها من المواد العاقة بها وفي مصر نعتمد على مياه نهر النيل والمياه الجوفية كمصادر رئيسية لمياه الشرب ، وتصل حصة مصر من مياه النيل إلى ٥٥.٥ مليار متر مكعب في السنة وذلك طبقا لإتفاقية عام ١٩٥٩ مع السودان.

## خصائص المياه

تنقسم خصائص المياه إلى :

١ -خصائص طبيعية

٢ -خصائص كيماوية

٣ -خصائص بيولوجية

### الخصائص الطبيعية

١. درجة الحرارة

٢. العكارة

٣. اللون

٤. الطعم

٥. الرائحة

**١-درجة الحرارة:**

تؤثر درجة الحرارة على عمليات تنقية المياه ، فهي تساعد على سرعة ذوبان الكيماويات المضافة وسرعة ترسب الجسيمات الدقيقة

**٢-العكارة :**

قد تكون مواد عضوية مثل الطحالب ومواد غير عضوية مثل الطمي والرمال . وتكون العكارة في المياه السطحية أكثر منها في المياه الجوفية نظراً لأن الأخيرة تتعرض للترشيح خلال مرورها في طبقات التربة المختلفة

**٣- اللون :**

يحدث تلون الماء في المورد السطحي نتيجة لتحلل المواد العضوية أو وجود مواد غير عضوية كالحديد والمنجنيز ، ويعتبر تلون الماء من أكثر الدلالات على عدم صلاحيته للاستهلاك الآدمي ومعظم الاستخدامات الصناعية.

**-الطعم:**

يكون للماء أحياناً طعم غي مستساغ ، وذلك نتيجة وجود طحالب ومواد عضوية متعفنة ، أو نتيجة اختلاطه بمياه الصرف أو المخلفات الصناعية قبل معالجته .

## الخصائص الكيماوية

١. الأس الايدروجيني
٢. العسر الكلي
٣. الأكسجين الذائب
٤. المواد الذائبة
٥. القلوية
٦. الحموضية
٧. المواد العضوية بأنواعها

**١. الأس الايدروجيني:**

وهو ما يرمز له بالرمز " pH "وهو يعبر عن الحالة الحمضية أو القلوية للماء ، وهو يبدأ من الصفر إلى رقم ١٤ ، والرقم ٧ يدل على التعادل النقي، وإذا قل الرقم عن ٧ يدل ذلك على حمضية الماء . فحينما تذوب أي مادة في المياه يتأين المحلول إلأى أيونات الهيدروجين +H وأيونات الهيدروكسيد OH ويكون الماء حمضي إذا كانت الأيونات +H أكثر من أيونات OH وقلوي إذا حدث العكس ، ويكون المحلول متعادلا إذا تساوي تركيز +H و -OH .

الماء الحمضي يحدث تأكلا في المعدات ، أما الماء القلوي فيرسب قشوراً.وقد وجد أنه بالنسبة للماء أو أي محلول مائي يكون حاصل تركيز أيونات الهيدروجين [ H+ ] والهيدروكسيد [ OH- ] يساوي مقدارا ثابتا هو ١× ١٠ -١٤ وذلك عند درجة حرارة 25م. أي أن 14-10×1= [ OH- ] [H+] .

وفي حالة المياه النقية يكون7-10= H+

وبالتالي7=[ 10-7 ] Log- =[ H+]log- =Ph

**٢. العسر :**

وهو عبارة عن وجود مقادير ملحوظة من أملاح الفلزات التي لا تذوب في الماء مثل الكالسيوم والماغنسيوم . ووجودها في الماء يزيد من الأس الأيدروجيني للماء ، والعسر يسبب قشورا داخل المواسير والعدادات وأجهزة تسخين المياه ، كما أ،ه يكسب الماء طعما غير مستساغ ويصعب معه استخدام الصابون.

**٣. الأكسجين الذائب:**

يتواجد الأكسجين ذائبا في المياه العذبة بصفة دائمة نتيجة للتهوية الطبيعية وتزداد نسبة الأكسجين الذائب في المياه الباردة عنها في المياه الساخنة ويؤدي وجود الطحالب في الماء إلى إنتاج الأكسجين نهاراً فيزداد منسوب الأكسجين الذائب في الماء وفي الليل تستنفذ الطحالب كمية من الأكسجين فينخفض منسوب الأكسجين الذائب في الماء وتساعد زيادة نسبة الأكسجين الذائب في الماء على حدوث التآكل في السطوح المعدنية الملامسة لها كالمواسير والعدادات والمضخات .

**٤. القلوية:**

تعزي قلوية المياه لوجود هيدروكسيدات–كربونات– بيكربونات ..بعض عناصر الفلزات النشطة (الاقلاء) مثل الصوديوم والكالسيوم و الماغنسيوم و البوتاسيوم ،وارتفاع قلوية المياه يؤدي إلى تزايد التكاثر البيولوجي وليست هناك أضرار من المياه المحتوية على قلوية حتى ٤٠٠ ملليجم /لتر.

**الحمضية :**

تكون المياه حامضية إذا كان الأس الهيدروجيني Ph أقل من ٧. ومن أسباب حمضية الماء وجود ثاني أكسيد الكربون الذائب أو بعض الأحماض العضوية الناتجة من تحلل البقايا النباتية كما أن تصريف المخلفات الصناعية التي تحتوي على أحماض في المسطحات المائية يزيد من درجة حمضية المياه ، وبالإضافة لما تسببه المياه الحامضية من تآكل وصدأ المواسير الحديدية فأنها تذيب بعض المواد الضارة بالصحة مثل النحاس والرصاص والزنك ، والمياه المفضل شربها تكون ( 6.5-8.5 ) PH.

٥. **المواد الذائبة :**

عند مرور المياه السطحية أو الجوفية على أنواع من التربة أو الصخور فأنها تذيب بعضا من هذه المواد الصلبة وتختلط بالماء ، وهناك حد أقصى مسموح به للمواد الصلبة الذائبة في الماء حتي لا تسبب للمستهلكين مشاكل صحية أو تكسب الماء طعما ورائحة غير مقبولين وتكون بعض المواد الذائبة ضارة بصحة الإنسان لذلك من الضروري إعطاء عناية للتخلص منها أثناء عمليات المعالجة .

٦. **المواد العضوية :**

تأتي نتيجة التلوث بالمخلفات السائلة الصناعية والزراعية والمجاري وهناك أنواع جديدة وكثيرة من المواد العضوية غير معروف تأثيرها في مياه الشرب على المدى الطويل إلا أن بعض هذه المواد مسببة لسرطان والبعض الآخر يغير في أساس تكون الخلايا .

## الخصائص البيولوجية

وهي عبارة عن ما تحويه المياه من بكتريا وفيروسات وطحالب وطفيليات ضارة بصحة الإنسان ، ويؤدي اكتشاف هذه البكتريا والفيروسات إلى وضع النظام السليم لتعقيم المياه بما يكفل قتل هذه الكائنات الدقيقة المسببة للأمراض .

## الاشتراطات الصحية في المياه

إن مسئولية القائمين على تشغيل محطات تنقية مياه الشرب التأكد من أن نوعية مياه الشرب المنتجة تتمشى مع المواصفات القياسية التي تكفل حماية الصحة العامة ، ويمكن تصنيف المواصفات القاسية إلى ثلاثة أنواع من الخواص هي :

### ١. الخواص الطبيعية:

**أ -درجات حرارة الماء:**

ليس لها تأثير على صحة العامة ولكن الارتفاع المفاجئ في درجة حرارة الماء قد يكون دلالة على تقلبات موسمية أو تصريف كبير لمياه الصرف الصحي أعلى التيار.

**ب -لون وطعم ورائحة الماء:**

هو من الاهتمامات الجمالية للمياه ولكن حدوثها قد ينبئ بحدوث تلوث من مصادر مثل المخلفات المنزلية أو الصناعية.

**ت -العكارة :**

هي من الأمور الهامة للصحة العامة لأنها تؤثر على عملية التطهير ، فالذي يحدث أنه طالما توجد جسيمات تعكر المياه فأنها تكون مخابئ تلجأ إليها الكائنات الحية للهروب من تأثير الكلورين عليها ، فلا يتم التطهير بالكامل وتصل هذه الكائنات إلى المستهلك وقد تصيبه بالأمراض ، لذلك حددت المواصفات القياسية أقصي مستويات للعكارة .

## ٢. الخواص الكيميائية :

تمثل الكيمياويات العضوية وغير العضوية خطراً على الصحة العامة ويوضح الجدول التالي تأثير الكيماويات المختلفة على الصحة العامة وأقصى مستوى مسموح به حفاظا على الصحة العامة .

**تأثير الكيماويات المختلفة على الصحة العامة**

| المادة | الأضرار الصحية المحتملة عند زيادة النسبة | أقصى نسبة مسموح بها(ملجم/لتر) |
|---|---|---|
| الزرنيخ | قروح على الأيدي والأقدام –ومسبب للسرطان –ومؤثر على الجينات على المدى الطويل –وأحياناً يمكن أن يسبب الإرهاق وفقدان الطاقة | ٠.٠٥٠ |
| الباريوم | ارتفاع ضغط الدم– تخدر الأعصاب | ١.٠٠٠ |
| الكادميوم | ارتفاع ضغط الدم –سام في حالة الاستنشاق –ومسبب للسرطان–وعلى المدى الطويل يتركز على الكبد والكلى والبنكرياس والغدة الدرقية | ٠.٠٠٥٠ |
| الكروم | حساسية في الجلد –فشل كلوي–والإصابة بالسرطان–وتأكل الأنسجة ويؤثر على المخ ويتلف الكلي | ٠.٠٥٠ |
| الرصاص | الأنيميا– شلل في الأطراف–إمساك –فقد الشهية | ٠.٠٥٠ |
| الزئبق | التهاب الفم– سقوط الأسنان– سام لجهاز العصبي على المدى الطويل | ٠.٠٠١ |
| السلينيوم | ضعف عام –تهيج الأنف والحلق–بقع حمراء بالأصابع | ٠.٠١٠ |
| الفضة | تحول لون الجلد إلرمادى–ويؤثر على العين والغشاء المخاطي | ٠.٠٥٠ |
| الألومنيوم | تؤثر على بعض أجزاء الجسم وخصوصا الكلي | ٠.٢٠ |
| الفلوريد | تبقع الجلد | ٠.٨٠٠ |
| النترات | ازرقاق لون الأطفال | ١٠.٠٠٠ |
| النتريت | ازرقاق لون الأطفال | ٠.٩ |

المعايير التي أقراتها اللجنه العليا للمياه في ١٩٩٥/٢/٢٦

### ٣. الخواص البيولوجية:

يمكن أن تنتقل الكائنات الحية المسببة للأمراض إلى الماء ، ومن أهمها بكتريا الكوليفورم ، وللكشف عن وجود الكوليفورم تستخدم إحدى طريقتين هما :

أ - **طريقة المرشح الغشائي:**

وفيها يجب ألا يتجاوز عدد الكوليفورم ١ لكل ١٠٠مل من العينة وذلك في عينة واحدة كمتوسط لجميع العينات المأخوذة خلال شهر واحد.

**ب -ب-طريقة التخمر بالأنابيب المتعددة:**

وفيها يجب أن ألا يتواجد عدد الكوليفورم عن ١٠٠/١ مل عينة لمرة واحدة طوال الشهر ولا يتجاوز أكثر من ٥% فقط من العينات خلال العام.

## أنواع المواد الدخيلة على المياه

**١)مواد ذائبة ( Dissolved Matters )**

وأهمها أملاح كربونات و بيكربونات وكبريتات و كلوريدات الكالسيوم و الماغنسيوم والصوديوم ، وكذا أملاح مركبات الحديد و المنجنيز و السيلكا ، هذا بالإضافة إلى فضلات المجاري والمصانع وعلاوة على الغازات الذائبة وأهمها الأكسجين وثاني أكسيد الكربون و كبريتيد الأيدروجين.

**٢)مواد عالقة (Suspended Matters )**

وأهمها الطين والرمل والمواد النباتية والحيوانية الميكروسكوبية وأنواع البكتريا ، وفضلات المصانع والصرف الصحي

**٣) -مواد كلويدية "غروية" (Colloidal Matters )**

وتوجد في حالة متوسطة بين التعلق والذوبان

# التأثيرات غير المرغوبة لبعض المواد الدخيلة على المياه

## المواد الذاتية

### أ – أملاح الكالسيوم و الماغنسيوم

البيكربونات: تسبب قلوية وعسر مؤقت

الكربونات : تسبب قلوية وعسر مؤقت

الكبريتات : تسبب عسر دائم

الكلوريدات: تسبب عسر دائم

### ب –أملاح الصوديوم

البيكربونات : تسبب قلوية

الكربونات : تسبب قلوية

الكبريتات : تسبب تكوين رغاوي في الغلايات

الفلوريدات : تسبب تشوية الأسنان

الكلوريدات : تسبب طعما

### ت –الغازات الذائبة

الأكسجين : تأثير على المعادن

ثاني أكسيد الكربون : تأثير على المعادن وحمضية

كبريتيد الأيدروجين : تأثير على المعادن وطعم ورائحة

## مواد عالقة

أ البكتريا : بعضها يسبب أمراضاً

ب – الطحالب: تسبب لوناً ، وطعماً ،ورائحة

ت – الطمي: يسبب عكارة

## مواد غروية

أ – أكسيد الحديد : يسبب لوناً أحمر

ب –المنجنيز: يسبب لوناً أسود أو بني

## المواد العضوية

تسبب لوناً وطعما ووجود بعض هذه المواد في المياه يجعلها غير نقية أو غير صالحة للاستعمال وقد تسبب الأمراض.

# الفصل العاشر

## أنظمة معالجة المياه

# الباب العاشر

# أنظمة معالجة المياه

نحصل على إمدادات الماء من الأنهار من الآبار أو من خلف السدود وتختلف نوعية الماء تبعاً لاختلاف المصدر الذي جاءت منه ، وبالتالي يحتاج كل نوع إلى معالجة خاصة حتى تكون المياه صالحة للشرب ففي مياه الأنهار وجد أنها تحتوي على شوائب صلبة عالقة علاوة على ما بها من تلوث ميكروبي ، أما مياه الآبار فتحتوي على مواد كيماوية ذائبة ويقل فيها التلوث الميكروبي أو ينعدم

## ١. معالجة المياه الجوفية

تختلف أنظمة معالجة المياه الجوفية والمياه السطحية تبغاً لاختلاف الملوثات المراد إزالتها, يتم معالجة المياه الجوفية إذا كانت تحتوي على مكونات تحول دون استخدامها مباشرة مثل أملاح الحديد و المنجنيز أو أملاح تسبب العسر مثل أملاح الكربونات و البيكربونات أو أملاح الكبريتات

و لا تخالف مكونات المياه الجوفية كثيرا من حيث الجودة ويكون اختلاف مكونات المياه الجوفية تبعا لاختلاف مصدر استخراج المياه الجوفية و يمكن تقسيم المياه الجوفية من حيث الجودة الي ثلاث مجموعات تبعا لطريقة معالجتها لإنتاج مياه الشرب

**أ – مياه جوفية هوائية**

**ب –مياه جوفية لا هوائية نسبيا**

**ت –مياه جوفية لا هوائية**

وهو ما يعني أن طريقة المعالجة تختلف تبعا لنسبة الأكسجين. وكذلك يمكن تصنيف نوع المياه الجوفية تبعا لتركيز الأكسجين أو الحديد أو المنجنيز .

مياه جوفية هوائية وهي المياه ذات المصدر المتصل بالهواء الجوي و عندما يكون المكونات العضوية للمياه محدودة لا تفقد المياه الأكسجين وبالتالي لا تحدث تفاعلات لا هوائية و في بعض الأحوال تكون المياه الجوفية الهوائية صالحة للشرب و بالتالي يمكن ضخها لشبكات مياه الشرب مباشرة , و علي الرغم من كون المياه الجوفية تحتوي علي

الأكسجين فان اولي خطوات المعالجة هي التهوية والغرض من هذه العملية زيادة تركيز الأكسجين وتقليل تركيز ثاني أكسيد الكربون و إذا كانت المياه المعالجة تلبي المواصفات القياسية المحلية يتم ضخها الي الشبكة فوراً.

العناصر التي يجب أن تؤخذ في الحسبان هي الأس الهيدروجيني PH والكالسيوم و SI و البيكربونات وهذه العناصر قد تخضع للمعالجة لجعلها تتناسب مع المواصفات القياسية .

## أ – المياه الجوفية الهوائية

عندما يكون مصدر المياه نربه رملية لا تحتوي علي الكالسيوم يكون المياه الجوفية المستخرجة عدوانية ويكون تركيز ثاني أكسيد الكربون اكبر من المطلوب وعليه يكون SI اصغر من الصفر يتم معالجة هذا بتهوية المياه وتقليل تركيز ثاني أكسيد الكربون ولكن يجب ان يتم بعد ذلك ضبط الأس الهيدروجيني و نسبة البيكمربونات بترشيح المياه علي فلتر من الحجر الجيري و حبيبات الرخام .( لتفي المياه بمتطلبات المواصفات القياسية )

*Figure 4 - Aeration above limestone filter*

الماء العسر هو الماء المستخرج من منطقة غنية بالكالسيوم تسمى معالجة المياه العسر ( softening) وهي عملية إمرار الماء العسر علي حبيبات من الرمل الناعم وحقن هيدروكسيد الكالسيوم أو هيدروكسيد الصوديوم ليحدث الترسيب كربونات الكالسيوم .

## ب –المياه الجوفية اللا هوائيه نسبياً

وهي المياه الجوفية المستخرجة من منطقة أسفل طبقة عازلة و بذلك تكون تحت ظروف نقص الأكسجين و وجود آمونيا ز الحديد و المنجنيز , ويتم معالجة هذه المياه بالتهوية

والترشيح من خلال وسط مسامي , والتهوية تعمل على إضافة الأكسجين و إزالة ثاني أكسيد الكربون وكذلك لأكسدة الحديد و المنجنيز وتحويل الامونيا الي نترات .

Figure 7 - Treatment of slightly anaerobic groundwa-ter

و خلال المرشح يتم حجز ندف الحديد المتكونة و علي سطحه تنمو بكتريا النيترة ويتم إزالة الامونيا بيولوجيا .

وفي مصر تستخدم هذه الطريقة في المعالجة التقليدية كما في شركة مياه بني سويف

**معالجة الحديد و المنجنيز – بني سويف ١**

**حيث يلي هدارات التهوية مرشح رملي مغمور .**

و توجد أيضا طرق أخري للمعالجة مثل التهوية خلال التربة وقبل استخراج المياه ( BERMAN ) وهي المطبقة في شركة مياه البحيرة وهي من الطرق المتقدمة والتي يلزم لها المتابعة الدقيقة والدراسة المتأنية قبل التطبيق

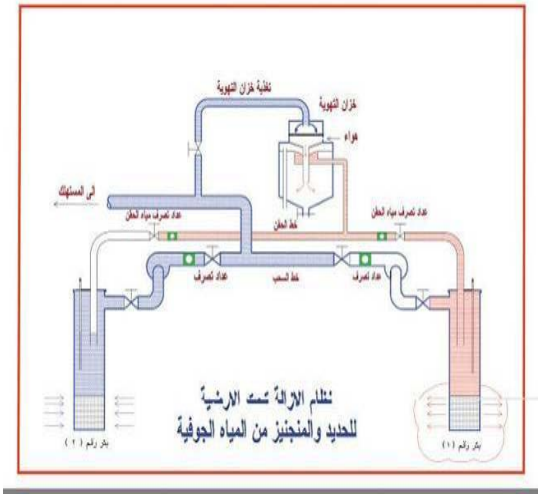
**وتتم عملية إزالة الحديد و المنجنيز بهذا الأسلوب على ثلاث مراحل:**

١- **الحقن** (Infiltration)، حيث يتم حقن كمية من المياه المهواة جيداً فى البئر المطبق به هذا الأسلوب ويتم تحديد الكمية طبقاً لأسس تصميم البئر وبالتجربة العملية بالموقع.

٢- **فترة الراحة** (Rest)، حيث يتم إيقاف الحقن وترك البئر دون سحب المياه لفترة زمنية.

٣- **السحب** (Abstraction)، يتم تشغيل البئر وضخ المياه المسحوبة إلى المناطق المخدومة بالبئر حتى الوصول إلى الحد الأقصى المسموح به للحديد والمنجنيز لتعاد الدورة من جديد.

وتعددت تفسيرات آلية إزالة الحديد و المنجنيز بأسلوب التحت سطحى إلا أن هناك تفسيراً أكثر شيوعاً يرتبط بوجود الحديد و المنجنيز فى صورة أيونات ذائبة بالمياه الجوفية فى ظل غياب الأكسجين، وأن تعرضها للأكسجين يؤدى إلى أكسدتها بعد إدمصاصها على سطح طبقة الأكاسيد المتكونة على حبيبات التربة لتخرج المياه من البئر خالية من هذه الأيونات..




**معالجة الحديد و المنجنيز – البحيرة ١**

## ت –<u>المياه جوفية اللا هوائية</u>

هي المياه المستخرجة من الأرض من خلال طبقة عازلة ولا يوجد أكسجين بها بل أيضا لا يوجد نترات و المواد العضوية متحللة , وتركيزات الحديد و المنجنيز و الامونيا عالية بها يتم معالجتها من خلال مرحلتي تهوية و مرحلتي ترشيح أولي مراحل الترشيح تتم من خلال مرشح جاف ثم يتبعه مرشح مغمور.

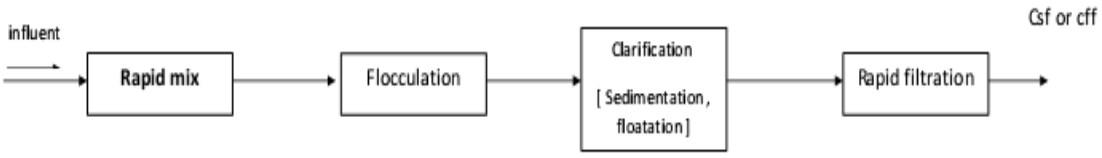
مخطط معالجة المياه اللاهوائية ١

## ٢.معالجة المياه السطحية

يتم معالجة المياه السطحية لتخليصها من الشوائب والمواد العالقة ومسببات البون والرائحة وتطهيرها من مسببات الأمراض و يعتبر نهر النيل والترع المصادر الأولي لإنتاج مياه الشرب في مصر نظراً لامتداد نهر النيل من الجنوب الي الشمال بفرعية رشيد ودمياط .

## أنظمة معالجة المياه السطحية

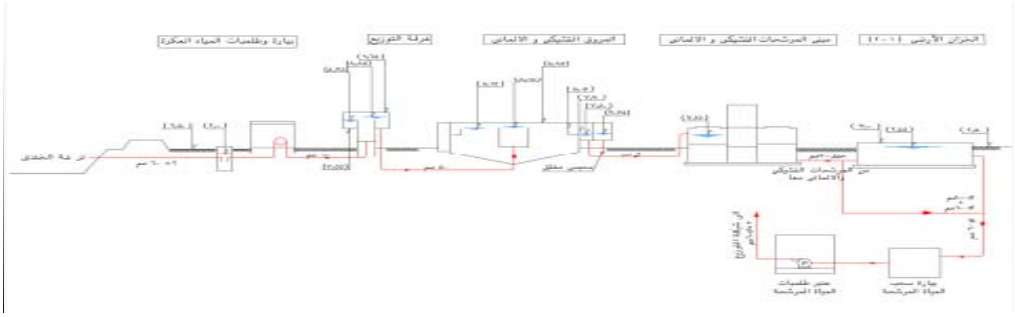
### أ – المعالجة التقليدية ( المزايا و العيوب )




مخطط المعالجة التقليدية للمياه السطحية 1

وهي المحطات التي تضم داخلها مجموعة من عمليات المعالجة الأساسية ابتداء من المصافي بالمدخل الي عملية حقن الكيماويات من الكلور والشبة والترويب ثم التنديف والترسيب والترشيح ثم التطهير والتخزين وهو ما سياتي تفصيلة لاحقاً , و تلك المحطات هي الأكثر انتشارا و الأكبر إنتاجا لمياه الشرب وذلك لما تتميز به من قدرة عالية علي مواهجة التغيرات الكبيرة في جودة المياه الخام وذلك نظرا لكبر المساحة وكمية الإنتاج وطول مدة مكث المياه داخل وحدات المعالجة المختلفة حيث قد تصل

مدة مكث المياه بالمحطة لفترات قد تتجاوز ٦ ساعات و هو ما يعطي فرصة اكبر للحد من التغير السريع وتأثر عملية المعالجة أو جودة المياه المنتجة .



**المروق – أهم مكونات محطة المعالجة ا**

و من المميزات أيضا لمحطات المعالجة التقليدية قدرة المشغل علي المناورة وتغيير أو إضافة مراحل معالجة جديدة كنقطة حقن جديدة للشبة أو الكلور أو تغيير معدل التحميل علي احد المرشحات أو المروقات والغرض هو المرونة العالية في التشغيل

و كذلك كميات الإنتاج الهائلة والقدرة علي تطوير المحطة وزيادة تحميلها للحصول علي كميات اكبر من المياه المعالجة في فترات الذروة وهو ما يحتاج الي طاقم تشغيل واعي ونشيط حي قان التحميل علي وحدات المعالجة قد يعطي انتاج اكبر ولكن يحتاج الي مراقبة مستمرة للحفاظ علي الجودة .

ومن الأنظمة المختلفة للمعالجة المطبقة بمحطات المعالجة التقليدية ما لا يحتاج الي مساحات كبيرة أنظمة الترويق باستخدام بطانية الروبة ذات معدل التحميل العالي وان اجتمع المصممين علي وضع قانون لتحديد المساحة التقريبية المطلوبة للمعالجة .

A= Q^0.7

where A in acers & Q in million galon per day

from { integrated design & operation of water treatment facilities} by KAWAMORA

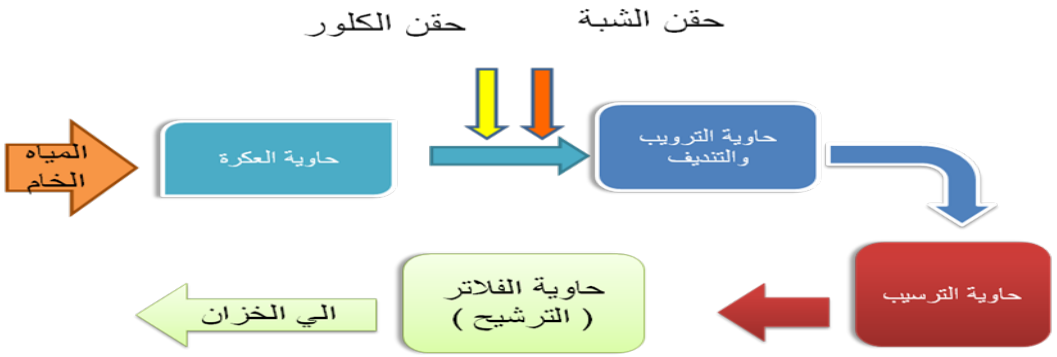
و المساحة الكبيرة المطلوبة لإنشاء محطة معالجة تقليدية هي احد عيوب محطات المعالجة التقليدية .

وكذلك الإعتمادات المالية الكبيرة والتي قد تقف مانعا لإنشاء المحطة وإمداد المواطنين بخدمة مياه الشرب هي ثاني عيوب هذه المحطات

أما الحاجة الي العمالة المدربة الواعية لتشغيل المحطة والحفاظ علي المنشئات فهي ضرورة منطقية للحفاظ علي الاستثمارات المبذولة في إنشاء محطة معالجة مياه الشرب .

و تكلفة التشغيل مثل تكاليف الكيماويات والكهرباء و مرتبات العاملين وتكاليف الصيانة و غيرها. يضع محطة المعالجة كأحد اكبر مشاريع الإنتاج

## ب –الترشيح السريع

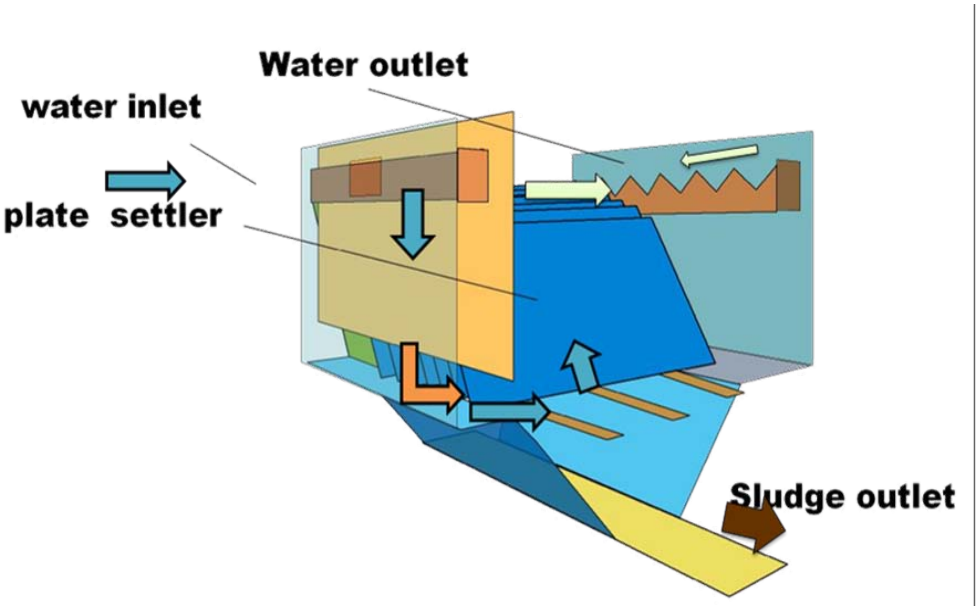



محطات الترشيح السريع هي وحدات متنقلة تستخدم لسد العجز في إنتاج المياه في المناطق التي لا تتوافر فيها المساحة المطلوبة لإنشاء محطة معالجة تقليدية و هذه المحطات هي وحدات ذات قدرات إنتاجية صغيرة ( ٣٠ لتر /ث ) وهي تخصص لخدمة قرية أو تجمع سكني صغير .

محطات الترشيح السريع ١

هذه المحطات تحتوي علي نفس مراحل المعالجة الموجودة بالمحطات التقليدية وان كانت مراحل المعالجة اصغر ومدة المياه بالمحطة اصغر وقد تصل مدة مكث المياه منذ دخولها المحطة الي خروجها الي الخزان حوالي ٢٠ دقيقة .

فبدلا من الترويب و التنديف بواسطة المندف الهيدروليكي ( الحواجز المتعاقبة ) يتم الترويب والتنديف بواسطة خلاط سريع وخلاط بطئ وقد يحل الحقن في ماسورة طرد طلمبات العكرة محل الخلط السريع و تحل مرشحات الضغط ( pressure filters ) محل المرشحات الرملية السريعة و يكون حوض الترسيب ذو الواح اللاميلا محل المروق التقليدي .



و تعتبر محطات الترشيح السريع هي احد المسكنات لشكوى نقص المياه والحالات الطارئة ونظرا لقصر مدة المعالجة فان المتابعة المستمرة لجودة المياه المنتجة ولكفاءة مراحل المعالجة من اهم شروط نجاح المعالجة بمحطات الترشيح السريع و قد يكون ضبط جرعة الشبة من أصعب العمليات و أكثرها أهمية ودقة .

## ت –الترشيح المباشر *direct filtration*

**مخطط المعالجة بالترشيح المباشر ١**

وهي عملية الترشيح التي يسبقها عملية تنديف في حوض منفصل ,وتتكون فيها ندف كبيرة (≤ ١ مم) ويتم إزالتها خلال مرشح ثنائي(ثلاثي ) الوسط ألترشيحي تكون الطبقة العليا فيه خشنة ويطلق أيضا علي هذه العملية ترشيح الندف floc filtration

**الترشيح المباشر – دمياط ١**

الترشيح بالتلامس contact filtration وفي هذه العملية لايوجد حوض منفصل للتنديف ويحدث عملية الترويب أثناء الخلط السريع وتثبيت الشحنات للمواد العالقة يحدث مباشرة قبل الترشيح ولا تتكون إلا الندف الصغيرة micro flocs

الترشيح بالتلامس يتميز عن الترشيح المباشر بتوفير التكلفة والمساحة الخاصة بمرحلة التنديف كما تستخدم كميات اقل من المواد الكيماوية حيث أنه في حالة الترشيح المباشر يجب ان تتكون ندف كبيرة

**مخطط المعالجة بالترشيح بالتلامس ١**

في المقابل فإنه في الواقع إن الاختلاف بين الترشيح بالتلامس والترشيح المباشر هو مجرد اختلاف نظري حيث يحدث التنديف في القنوات والمواسير التي تسبق المرشحات ولدلك يدخل المرشحات خليط من الندف الصغيرة ( micro flocs ) والندف المتجمعة الأكبر نسبيا .

بل قد يتحول الترشيح بالتلامس إلى ترشيح مباشر بسيط عندما تتكون الندف بصورة غير مقصودة مثلا في المدة التي يقضيها الماء اعلي الوسط ألترشيحي , الترشيح بالتلامس يتطلب مراقبة دقيقة وحذرة حيث أن عملية المعالجة حساسة جدا لتغير جودة المياه الخام , إن عدم وجود حوض تنديف (يعمل علي استقرار ظروف تغير المياه الخام ) هو احد عيوب الترشيح بالتلامس. بل أكثر من هذا انه يصبح من الصعب الحصول علي مياه مرشحة ذات عكارة تساوي ٠.١٥ وحدة عكارة في حالة معدلات الترشيح العالية .

الترشيح المباشر ينحصر استخدامه في معالجة المياه الخام ذات العكارة وتركيز الطحالب المتوسطين ولا يوجد العديد من المياه الخام التي تناسب هذه المواصفات .

الترشيح المباشر مناسب فقط لمعالجة للمياه ذات العكارة (١٠وحدات عكارة ) ولا تزيد عن {٢٥ وحدة عكارة و ٤٠ هازن (وحدة قياس اللون ) في أسوا الظروف }

إن المياه ذات العكارة واللون الأعلى قد يمكن معالجتها أيضا باستخدام الترشيح المباشر ولكن يحتاج الأمر إلي عمل محطة تجريبية لكل حالة علي حدة .

وبالنسبة للكائنات الحية العالقة فهي أيضا احد العوامل المحددة لاستخدام الترشيح المباشر فمما لاشك فيه إن الهجوم الطحلبي في المياه المفتوحة يؤثر عكسيا علي جودة

المياه الخام وبالتالي علي أداء المرشحات من حيث مدة تشغيل المرشح وجودة المياه المنتجة , وانه لمن الصعب وضع قيم محددة للعدد الطحلبي الذي يمكن معه استخدام الترشيح بالتلامس او الترشيح المباشر حيث يعتمد أداء المرشح بصورة أساسية علي الوسط ألترشيحي المستخدم , ولكن توجد تقارير تفيد أن العدد الطحلبي حتى ٢٠٠٠ /مل تم تحديده كقيمة قصوى .

أن الترشيح بالتلامس والترشيح المباشر يمكن أن يتغلب علي مشكلات الطحالب حتى تركيز كلوروفيل A لا يزيد عن ١٠ميكور جرام /لتر, إن الخبرات العملية التي أجريت بألمانيا تشير الي أن تركيز الكلوروفيل A ٥٠ ميكروجرام /لتر يمكن قبوله تحت ظروف معينة .

و بخلاف الشروط الكثيرة المحددة لاستخدام الترشيح بالتلامس أو الترشيح المباشر فان قلة التكلفة و صغر جرعة الشبة المطلوبة وصغر المساحة المطلوبة لإنشاء محطة تعمل بالترشيح المباشر احد أهم مميزات عملية المعالجة وقد يلجا البعض لاختيار طريقة المعالجة بالترشيح المباشر متجاهلاً إمكانية تغير جودة المياه الخام مع الوقت أو تحت ظروف انخفاض منسوب المياه بالمأخذ .

# الفصل الحادي عشر

## عمليات معالجة مياه الشرب

# الباب الحادى عشر

# عمليات معالجة مياه الشرب

## أولاً :فصل الاجسام الطافية والعالقة بالمأخذ :

هي أولى عمليات المعالجة ونعني بها فصل المواد الصلبة و الإعشاب الطافية بالماء و يتم ذلك بواسطة المصافي بمأخذ المحطة , وتكون المصافي إما ميكانيكية أو يدوية و يتم الاهتمام بتنظيفها يوميا لضمان عدم انسداد الماخذ و توقف تدفق المياه الخام الي المحطة .

والمصافي تكون علي صفين احدهما تلو الأخر الأولى ذات مسافات بينية بين القضبان اكبر من الصف الثاني ليتم توزيع الأجسام العالقة بين كلا الصفين

# ثانيا:عمليتي الترويب و التنديف

## طبيعة الشوائب

تتكون الشوائب الموجودة في المياه نتيجة التآكل الأرضي الناتج من احتكاك المياه مع قطاع المجري المائي ، وتتكون الشوائب من دقائق صغيرة كما تلتقط المياه مع الاحتكاك أيضا كميات من الأملاح وبقايا النبات المتحللة كما توجد شوائب أخرى إضافية ناتجة من التلوث الهوائي ومخلفات المصانع وفضلات الحيوانات ، ولذلك فالمياه السطحية تتلوث بتأثير الطبيعية والإنسان وتحتوى المياه على مواد عالقة ومواد ذائبة عضوية من أصل نباتي أو حيواني ) وكذلك على المواد غير العضوية وكافة الأشكال البيولوجية كالبكتريا والنباتات المائية والهائمات (plankton) وتسمى هذه المواد " المواد العالقة الصلبة " وتكون قدراً كبيراً من مكونات الشوائب ، أما الأجزاء الكبيرة الحجم مثل الرمل و الطمي الثقيل "Heavy Silt" فيمكن إزالتها من المياه بتركها للترسيب بواسطة الجاذبية الأرضية وتسمى هذه المواد " المواد الصلبة القابلة للترسيب (Settelable Solids) . ولكل منها حجمها وتأثيرها المختلف على المياه.

يمكن تقسيم الشوائب بالمياه أيضا تبعا للحجم فنجد أن :

١) مواد عالقة ( suspended solids ) وهي ذات أقطار > $١٠^{-٦}$ م

٢) $١٠^{-٦}$ م > مواد غروية (colloidal solids ) > $١٠^{-٩}$ م

٣) مواد ذائبة (dissolved solids ) < $١٠^{-٩}$ م

و هو مما يجعلأيضا لكل منها سرعة ترسيب مختلفة .

| Diameter (m) | Types of particles | Settling time over 30 cm |
|---|---|---|
| $1.10^{-2}$ | gravel | 0.3 sec |
| $1.10^{-3}$ | coarse sand | 3 sec |
| $1.10^{-4}$ | fine sand | 38 sec |
| $1.10^{-5}$ | silt | 33 min |
| $1.10^{-6}$ | bacteria | 35 hours |
| $1.10^{-7}$ | clay | 230 days |
| $1.10^{-8}$ | colloids | 63 years |

**الجدول السابق يوضح سرعة ترسيب مختلف المواد لأقطار مختلفة**

و للتعبير عن كمية المركبات الموجودة بالمياه يوجد العديد من العوامل المستخدمة مثل تركيز المواد العالقة, العكارة ( TOC / DOC ) واللون

ويحدث ترسيب للمواد الصلبة طبيعيا عند تخزين المياه فترة كافية من الزمن في خزان أو في الأنهار والبحيرات ذات السرعة البطيئة ، لكن الأجزاء الصغيرة جداً مثل المواد الغروية والطمي الدقيق والبكتريا التي لا تترسب بسهولة وتحتاج إلى معالجة لتجعل جزيئاته أكبر وذات قابلية للترسيب وتسمى هذه المواد بالمواد غير قابلة للترسيب أو المواد الغروية المواد الغروية هي سبب وجود العكارة وهي ذات اقطار صغيرة وتحمل شحنة سالبة وتتنافر فيما بينها , في المناطق الاستوائية يمكن ان يزداد تركيز المواد العالقة و يصبح النهر عبارة عن سريان للروبة .

تسبب المواد الذائبة في وجود اللون للمياه الخام وإزالة هذه المواد يعتمد اعتمادا كبيرا علي الأس الهيدروجيني للمياه

*Figure 2 - Rivers in the tropics sometimes have high suspended solids contents*

**الحاجة إلى الترويب والتنديف**

الغرض من هاتين العمليتين هو إزالة المواد الشائبة وخاصة غير القابلة للترسيب الطبيعي وإزالة العكارة من المياه المراد معالجتها .

وتعمل المواد الكيماوية (المروبات) على إزالة المواد الغير قابلة للترسيب بعملية الترويب (Coagulation ) وذلك بأن تجعل هذه الجزيئات تتجمع مع بعضها البعض (Clump together) لتكون الندف الصغيرة ثم تتجمع الندف الصغيرة مع بعضها البعض لتكوين ندف كبيرة وثقيلة قادرة على الترسيب ، و باختصار فإن عملية الترويب هي عملية إضافة مروب لمياه الخام لعمل (destabilization ) للجزيئات ذات الشحنات السالبة.

فتخلط المواد الكيماوية بالماء العكر الذي يحتوي على جسيمات دقيقة لا ترسب ولا تترشح بسهولة وتعمل على جعل الجسيمات الدقيقة الشديدة الصغر تتجمع وتتكتل معا في صورة أجسام أكبر (ندف)

للحديث عن آلية الترويب يمكن القول ان الترويب يحدث بثلاثة آليات مختلفة

1. electrostatic coagulation
2. adsorptive coagulation
3. Precipitation coagulation

**الترويب الاليكتروستاتيكي**

وفي الترويب الاليكتروستاتيكي تقترب الايونات الموجبة ومن الطبقة المحيطة بالمواد الغروية ذات الشحنة السالبة كما بالشكل والجانب الأيسر قبل حقن المروب والجانب الأيمن بعد حقن المروب وهذا النوع من الترويب يحدث بعد حقن 0.025 mol/m3 من الايونات المنتقلة وهذا لا يوجد في المياه السطحية ولذا لا يلعب هذا النوع من الترويب دورا هاما في معالجة المياه السطحية

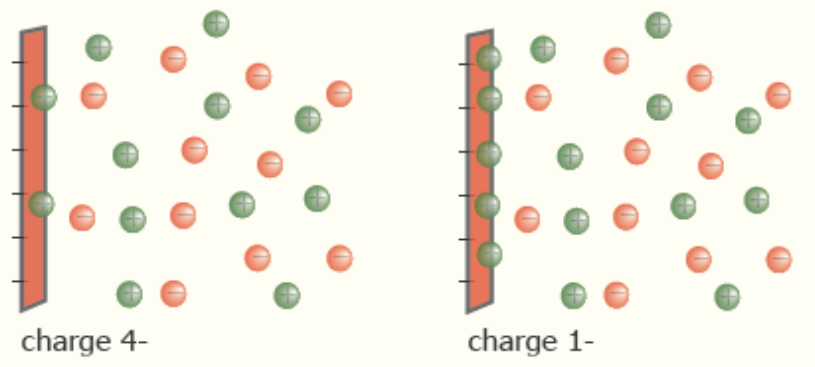



*Figure 6 - Mechanism of electrostatic coagulation*

**الترويب الامتزازي ( adsorptive coagulation )**

و في هذا النوع من الترويب تمتز جزيئات المواد الغروية الي الجزيئات الموجبة المتكونة من تفاعل المروب مع الماء FeOH2+ and FeOH2+ وهذا النوع من الترويب لايحدث الا في حالات PH المنخفضة بالنسبة لكلوريد الحديديك أفضل PH من ٦ الي ٨ و في حالة الشبة ( ايون الألمنيوم فإن ال PH المثلي ٧ و من خصائص خذا النوع من الترويب ان نسبة ازالة المواد العضوية يتناسب مع كمية المروب المضافة و يمكن ان يحدث

إعادة تنافر الجزيئات عند وبادة الجرعة عن الحد المطلوب.و زمن حدوث هذه العملية ثانية واحدة فقط

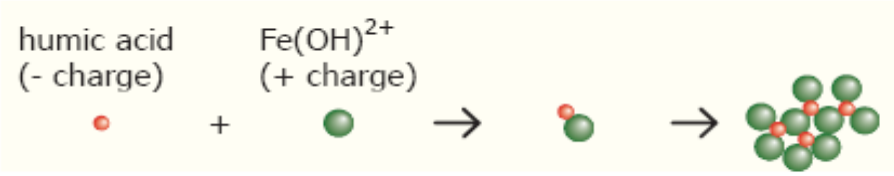



*Figure 7 - Mechanism of adsorptive coagulation*

## الترويب الترسيبي Precipitation coagulation

وهذا النوع من التريب يحدث في المياه ذات الماد العالقة المنخفضة حيث تتحد المواد الغروية الي ندف متعادلة وهذا النوع من الترويب يحتاج الي جرعات اعلي من المادة المروبة اكبر من المضافة الي الترويب بالامتزاز .

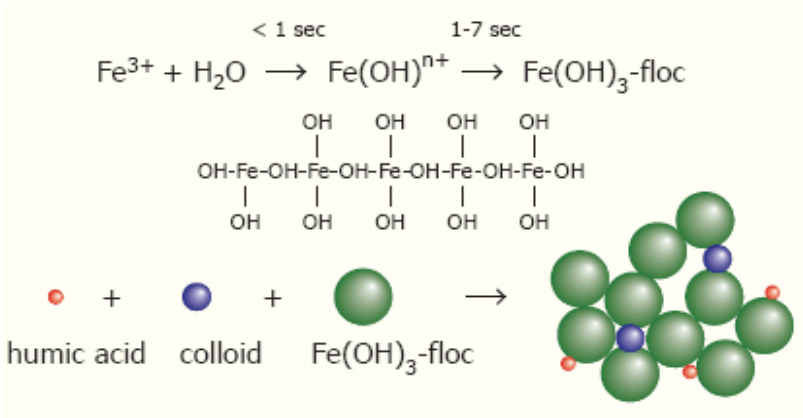



*Figure 8 - Mechanism of precipitation coagulation*

## مواد الترويب

من المواد المستخدمة في الترويب

١. كلوريد الحديديك , ويتميز بقدرتة الجيدة علي العمل في مدي كبير من الاس الهيدروجيني PH حيث يعمل في وسط الاس الهيدروجيني له من ٦ الي ٩

٢. كبريتات الالمونيوم و اسمها الدارج الشبة وهي تتميز بتوفرها ورخص ثمنها و تعمل في وسط الاس الهيدروجيني له من ٥.٥ الي ٧.٥ .

## عملية الترويب

هي عملية الخلط السريع للمياه الخام مع المادة المروبة و تستغرق هذه العملية من ١ الى ٧ ثواني والهدف منها توزيع المادة المروبة على كل كمية المياه الخام و يستلزم لذلك بذل طاقة للخلط ويتم ذلك من خلال خلاط ميكانيكي أو بواسطة الخلط الهيدروليكي كما يمكن ان يحدث الخلط السريع بحقن المروب في خط طرد طلمبات المياه العكرة.

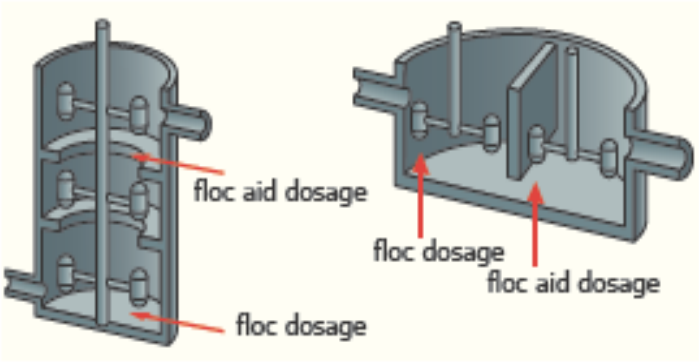



*Figure 16 - Mechanical mixers*

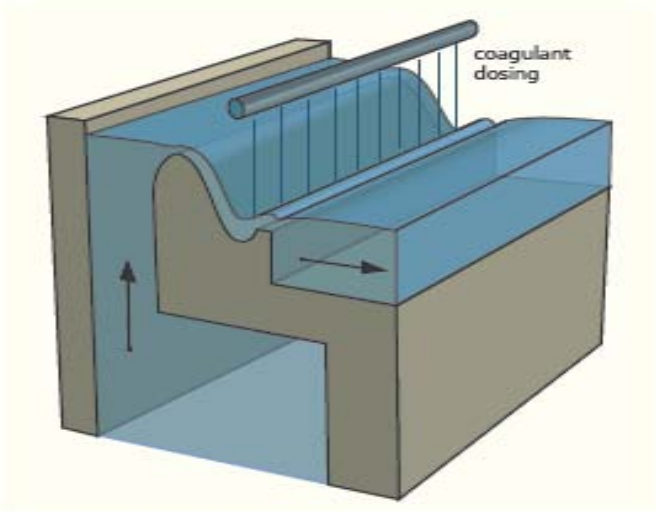



*Figure 17 - Cascade mixer*

و لكمية الطاقة المبذولة في الخلط تأثير علي جودة عملية الخلط ولها حدود يجب التأكد منها لضمان جودة هملية الترويب .

وهذه القيمة تسمى G value

$$G_c = \sqrt{\frac{P}{\mu \cdot V}}$$

| | | | |
|---|---|---|---|
| $G_c$ | = | velocity gradient for rapid mixing | $[s^{-1}]$ |
| P | = | dissipated power | [W] |
| $\mu$ | = | viscosity | $[N \cdot s/m^2]$ |
| V | = | volume of mixing tank | $[m^3]$ |

وهذه القيمة تعتمد على لزوجة المياه الخام , حجم حجم الحوض الذي يحدث به الخلط , و قدرة المحرك المستخدم في الخلط .

وفي حالة الخلط الهيدروليكي تكون المعادلة نعوض عن الطاقة كما يلي :

$$P = \rho \cdot g \cdot Q \cdot \Delta H$$

$$G_c = \sqrt{\frac{\rho \cdot g \cdot \Delta H}{\mu \cdot \tau_c}}$$

وكما نري ان قيمة ال G value تعتمد على فرق الارتفاع بين دخول وخروج المياه من حوض الترويب و تصرف المياه العكرة .

وقيمة ال G value للترويب علي الأقل ١٥٠٠ ث$^{-١}$

## التأكد من جرعة الشبة

يمكن التأكد من جرعة الشبة المضافة بواسطة قياس التغير في القلوية للمياه الخام , وهذه الطريقة تقريبية و تعتمد علي العامل البشري في قياس القلوية وكذلك على جودة المواد الكيماوية المستخدمة .

١. يتم قياس قلوية المياه العكرة

a. التأكد من جرعة الكلور المبدئي المستخدمة جم/م٣

٢. يتم حساب التغير في القلوية حيث حيث يعمل كل ١ جم كلور علي تخفيض القلوية بمقدار ٠.٦

٣. يتم حساب القلوية بعد إضافة الكلور

= قلوية المأخذ – ( جرعة الكلور جم/م٣ ×٠.٦ )

٤. يتم قياس القلوية قبل دخول المروق

٥. يتم حساب فرق القلوية نتيجة إضافة الشبة

٦. الجرعة = ( ١٠٠ × فرق القلوية )/٤٥ ( في حالة الشبة الصلبة )

الجرعة = فرق القلوية × ٤.٤ ( في حالة الشبة السائلة )

# التنديف Flocculation

## طرق التنديف

عملية التنديف هي عملية مزج بطيء للمياه المروبة و الهدف منها تجميع الجزيئات المروبة لتكوين الندف – و الشكل لأمثل للندف في حجم رأس الدبوس من ١ : ٣ مم – استعدادا لعملية الترويق .

تتم عملية التنديف بواسطة خلا ميكانيكي بطيء داخل حوض التنديف كما بالشكل

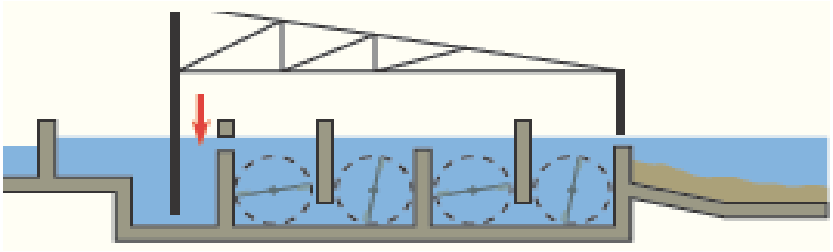


*Figure 22 - Floc formation installation WRK I/II*

أو من خلال خلط هيدروليكي للمياه يتم الخلط فيه من خلال سريان الماء بين حواجز كما بالشكل و في هذا النوع من أحواض التنديف تكون السرعة في حدود ٠.٣ م/س و مدة المكث في حدود ٢٠ الي ٣٠ دقيقة .

*Figure 25 - Hydraulic floc formation*

و من التصميمات المستخدمة للتنديف هي حوض التنديف داخل المروق الداثري ويحدث الخلط بواسطة قلاب كبير يدور بسرعة بطيئة داخل حوض التنديف في وسط المروق الدائري ويطلق علي هذا التصميم ( clari flocculator )كما هو موضح بالشكل .

و لعملية التنديف ايضا شروطها ومنها ان مدة المكث داخل حوض التنديف في حدود من ٢٠ :٣٠ دقيقة و الطاقة المبذولة في الخلط لها قيمة محددة ( G value ) و يمكن حساب بنفس قوانين الترويب و قيمتها تتراوح من ٢٠:٧٥ ث $^{-1}$ .

## العوامل المؤثرة على التنديف

أ – أولا الأس الهيدروجيني حيث يفضل أن تعمل الشبة في وسط يتراوح قيمة ال PH من ٥.٥ الي ٧.٥

ب –قيمة ال G value كما سق الذكر .

ت –جرعة الشبة المستخدمة ويتم تحديدها بواسطة ال jar test كما سياتي تفصيلا.

ث –عكارة المياه الخام , حيث يصعب تنديف المياه قليلة العكارة

## الترويب والتنديف عمليا

للبدء في أعمال الترويب والتنديف يجب أولاً العناية بتداول وتخزين المواد المروبة ثم بعد ذلك البدء في تحضير المحاليل الخاصة منها ليتم إضافتها للمياه حتى تتم عملية الترويب والتنديف بكفاءة ، لذلك سنتعرض في هذا الفصل إلى التجهيزات الخاصة بعملية الترويب /التنديف من حيث تخزين الكيماويات ، وتحضير المحاليل .

**تخزين الكيماويات**

تتوفر المروبات في شكل صلب (مسحوق أو حبيبات ) أو سائل وغالباً ما تكون السوائل أسهل في الخلط والاستعمال ويمكن شراء الكيماويات بأحجام مختلفة تتراوح من أكياس زنة ٢٥ كيلو جرام وأوعية سعة ٢٠ لتر إلى سيارات نقل صهريجية زنة ٥٠ طن أو أكثر.

ويجب دائما تخزين الكيماويات في منطقة جافة عند درجة حرارة متوسطة ومنتظمة إلى حد ما ، حيث أن الرطوبة قد تسبب تصلد أو تجبس للكيماويات الجافة فمن الواجب تخزينها على "طبالي" خشبية نقالة للسماح بمرور الهواء أسفل الأكياس ، ومن المفضل أيضا تخزين الكيماويات الجافة على أرضية تقع فوق منطقة تحضير المحاليل حتى يمكن تغذيتها بالجاذبية إلى المنطقة الخاصة بالتحضير ، وعلى أية حال فقد لا يكون ذلك عمليا في المحطات الصغيرة والمتوسطة الحجم أن حجم الكيماويات المستعملة قد لا يعتبر استثمارا رئيسيا في معدات تداول الكيماويات .

وتشمل المعدات المستعملة في تداول الكيماويات العربات اليدوية أو الميكانيكية والأوناش العلوية والمصاعد والناقلات الميكانيكية والناقلات بالهواء المضغوط وتتباين اشتراطات التخزين والتداول وفقا لنوع ومقدار المادة الكيمائية المستعملة

وتخزن الكيماويات غالبا بحالة جافة أو سائلة في منطقة تخزين خاصة ومحمية الجوانب وتتاح التعليمات عن التخزين المناسب للمروبات من مورد الكيماويات كما يمكن الاطلاع على الطرق السليمة لتداول وحفظ الكيماويات من خلال المواصفات القاسية المصرية الخاصة بالتداول والتخزين بكل نوع من هذه المروبات

## تحضير المحاليل

يتم عادة تخفيف المروبات في محطة المعالجة إلى محاليل ذات تركيز مناسب قبل إضافتها إلى المياه الخام وغالبا ما يكون التركيز ١٠%.

وفي المحطات الضغيرة يمكن إجراء ذلك عن طريق تفريغ الأكياس يدويا أو التخفيف لتحضير خلطا متساويا ومتجانسا في الماء ، وفي المحطات الكبيرة يخصص مبنى كامل للتحضير الاتوماتيكى لمحاليل الترويب وقد يشمل مثل هذا المبنى على قواديس كبيرة وموازين وناقلات بسيور وأوناش وشبكات من المواسير وطلمبات لمعايرة الكيماويات وصهاريج للتخزين المؤقت

ويتكون محلول الشبه من محلول حمضي آكل يتفاعل من المادة الحاوية له ، لذلك يتم إنشاء أحواض تجهيز المحلول ممن المباني أو الخرسانة ثم يتم تبطينها بادة لا تتأثر بالأحماض مثل السيرميك أو تدهن بمواد لا تتأثر بالأحماض مثل الإيبوكسي والفيبرجلاس (الألياف الزجاجية).

## تحضير المحاليل االمخففة من الشبه

يتم إضافة مائة كيلو جرام من الشبه الصلبة (شيكارتين زنة الواحدة ٥٠ كجم ) لكل متر مكعب من المياه في الحوض لتعطي تركيز ١٠% أما في حالة الشبه السائلة (٥٠%) ذات كثافة ١.٣٢ فيتم التخفيف إلى ٢٠% من وزنها (ما يعادل ١٠% صلبة).

مثال : حوض أبعاده ٢× ٢× ٦ متر (ارتفاع)

سعة حوض التخفيف ٢٤م٣

كثافة الشبه ٥٠% = ١.٣٢

كتلة الشبه في الحوض كصلبة (١٠٠%)= ١٠٠×٢٤/ ١٠٠٠ = ٢.٤٠٠طن

كتلة الشبه في الحوض كسائلة (٥٠%)=٢.٤× ٢=٤.٨ طن

حجم الشبه في الحوض كسائلة (٥٠%)=٤.٨/١.٣٢ = ٣.٦٣م٣

أي يضاف ٣٦٣٦ لتر في حوض سعة ٢٤م٣ من الشبه (٥٠%) لتعطي تركيز ١٠% وهو يعادل =٣٦٣٦ ×١.٣٢= ٤.٨ طن .

| حجم محلول الشبه م٣ | ارتفاع المياه | حجم المياه | كمية الشبه /طن | حجم الشبه م٣ |
|---|---|---|---|---|
| ٢٤ | ٥.٠٩١ | ٢٠.٣٦٤ | ٤.٨ | ٣.٦٣٦ |
| ٢٢ | ٤.٦٦٦ | ١٨.٦٦٧ | ٤.٤ | ٣.٣٣٣ |
| ٢٠ | ٤.٢٤٢٥ | ١٦.٩٧٠ | ٤ | ٣.٠٣٠ |
| ١٨ | ٢.٨١٨ | ١٥.٢٧٣ | ٣.٦ | ٢.٧٢٧ |
| ١٦ | ٣.٣٩٤ | ١٣.٥٧٦ | ٣.٢ | ٢.٤٢٤ |
| ١٤ | ٢.٩٦٩ | ١١.٨٧٩ | ٢.٨ | ٢.١٢١ |
| ١٢ | ٢.٥٤٥ | ١٠.١٨٢ | ٢.٤ | ١.٨١٨ |
| ١٠ | ٢.١٥١ | ٨.٤٨٥ | ٢ | ١.٥١٥ |
| ٨ | ١.٦٩٧ | ٦.٧٨٨ | ١.٦ | ١.٢١٢ |
| ٦ | ١.٢٧٢ | ٥.٠٩١ | ١.٢ | ٠.٩٠٩ |
| ٤ | ٠.٨٤٨ | ٣.٣٩٤ | ٠.٨ | ٠.٦٠٦ |

**جدول يوضح كمية وحجم الشبه السائلة اللازمة لتحضير محلول تركيزه يعادل ١٠% صلبة لارتفاعات مختلفة من الحوض .**

ويتم إتباع الخطوات التالية لتجهيز المحلول المطلوب :

يتم إضافة مياه للحوض من المحبس وعندما يصل منسوب المياه لمستوي ريش الخلاط يتم تشغيله لتقليب المحلول .

يغلق محبس إضافة المياه للحوض عند ملئه .

استمر في التقليب حتى إذابة كل الشبه .

إبلاغ المعمل لأخذ عينة في اليوم التالي.

يتم السحب من حوض الشبه حتى يصل إلى مستوي عوامة أقل منسوب .

يجب مراجعة تركيزالشبة دوريا باستخدام الهيدروميتر و التاكد من معايرتة واتقان الفنيين استخدامه

تحضير محلول شبة يعادل تركيز ١٠ % صلبة باستخدام الشبة السائلة ذات تركيز ٥٠ %*

| ارتفاع محلول الشبة المطلوب تجهيزه (م) | ارتفاع المياه المضافة (م) | حجم المياه (م٣) | ارتفاع الشبة ٥٠% المضافة (م) | حجم الشبة ٥٠% (م٣) |
|---|---|---|---|---|
| ٤ | ٣,٣٩٤ | ١٣٠,٦٦٩ | ٠,٦٠٦ | ٢٣,٣٣٣ |
| ٣,٩ | ٣,٣١ | ١٢٧,٤٣٥ | ٠,٥٩٠ | ٢٢,٧٥ |
| ٣,٨ | ٣,٢٢٥ | ١٢٤,١٦٢ | ٠,٥٧٥ | ٢٢,١٦٦ |
| ٣,٧ | ٣,١٤ | ١٢٠,٨٩ | ٠,٥٦٠ | ٢١,٥٨٣ |
| ٣,٦ | ٣,٠٥٥ | ١١٧,٦١٧ | ٠,٥٤٥ | ٢٠,٩٩٩ |
| ٣,٥ | ٢,٩٧ | ١١٤,٣٤٥ | ٠,٥٣٠ | ٢٠,٤١٦ |
| ٣,٤ | ٢,٨٨٥ | ١١١,٠٧٢ | ٠,٥١٥ | ١٩,٨٣٣ |
| ٣,٣ | ٢,٨٠ | ١٠٧,٨ | ٠,٥٠٠ | ١٩,٢٤٩ |
| ٣,٢ | ٢,٧١٥ | ١٠٤,٥٢٧ | ٠,٤٨٥ | ١٨,٦٦٤ |
| ٣,١ | ٢,٦٣ | ١٠١,٢٥٥ | ٠,٤٧٠ | ١٨,٠٨٣ |
| ٣ | ٢,٥٤٥ | ٩٧,٩٨٢ | ٠,٤٥٥ | ١٧,٤٩٩ |
| ٢,٩ | ٢,٤٦١ | ٩٤,٧٤٨ | ٠,٤٣٩ | ١٦,٩١٦ |
| ٢,٨ | ٢,٣٧٦ | ٩١,٤٧٦ | ٠,٤٢٤ | ١٦,٣٣٣ |
| ٢,٧ | ٢,٢٩١ | ٨٨,٢٠٣ | ٠,٤٠٩ | ١٥,٧٤٩ |
| ٢,٦ | ٢,٢٠٦ | ٨٤,٩٣١ | ٠,٣٩٤ | ١٥,١٦٦ |
| ٢,٥ | ٢,١٢٢ | ٨١,٦٩٧ | ٠,٣٧٨ | ١٤,٥٨٣ |
| ٢,٤ | ٢,٠٣٦ | ٧٨,٣٨٦ | ٠,٣٦٤ | ١٣,٩٩٩ |
| ٢,٣ | ١,٩٥٢ | ٧٥,١٥٢ | ٠,٣٤٨ | ١٣,٤١٦ |
| ٢,٢٠ | ١,٨٦٧ | ٧١,٨٧٩ | ٠,٣٣٣ | ١٢,٨٣٣ |
| ٢,١ | ١,٧٨٢ | ٦٨,٦٠٧ | ٠,٣١٨ | ١٢,٢٤٩ |
| ٢ | ١,٦٩٧ | ٦٥,٣٣٤ | ٠,٣٠٣ | ١١,٦٦٦ |
| ١,٩ | ١,٦١٢ | ٦٢,٠٦٢ | ٠,٢٨٨ | ١١,٠٨٣ |

* كثافة الشبة السائلة الخام ١,٣٢ جم / سم٣ في هذا المثال.

## تحديد جرعة الشبه الصحيحة

للبدء في إجراءات التحكم في التشغيل وتحديد إضافة الشبه ، فأنه من الضروري البدء أولاً في إجراء تجربة اختبار تحديد الجرعات (Jar Test ) والتي يتم على ضوء نتائجها تحديد جرعة الشبه المطلوبة إضافتها للمياه.

## جهاز تحديد الجرعات(Jar Test )

يتكون الجهاز كما هو مبين الشكل رقم (٥-١) من مجموعة من الكؤوس الزجاجية والذي يستخدم لتحديد جرعة الشبه الفعالة معملياً

*Figure 9 - Jar-test apparatus*

ويوجد في كل كأس قلاب صغيرة ويتم إدارة هذه القلابات بواسطة عامود إدارة عن طريق تروس ، ويتم إدارة العامود بواسطة محرك كهربائي صغير .

ويتم دوران القلاّبات عن طريق التروس ، بسرعة تماثل سرعة تحرك الماء في المروقات ويتم التقليب السريع بسرعة ٨٠-٣٠٠ لفة /دقيقة لمدة (١٥-١٢٠)ثانية وتخفض السرعة إلى(٢٠-٤٠) لفة /دقيقة لمدة(١٥-٤٥) دقيقة بعد إضافة الجرعات المختلفة .

## خطوات تجربة تحديد الجرعات (Jar Test )

يتم اتباع الخطوات التالية لإجراء تجربة تحديد جرعات الشبه المطلوب إضافتها للمياه ونبدأ أولا بتجهيز محلول الشبه قبل متابعة باقي خطوات التجربة

**تجهيز المحلول**

- **في حالة الشبه السائلة :**

ضع ١٥.٦ مللي لتر من الشبه السائلة (كثافة نوعية ١.٣٢٥) في قارورة سعة ١٠٠٠ مللي لتر وأكمل القارورة حتى علامة ١٠٠٠ بالماء لنحصل على محلول شبه بتركيز ١٠٠٠٠مجم /لتر (١%)

- **في حالة الشبه الصلبة :**

ضع ١٠ جم من الشبه الصلبة سلفات الألومونيوم ****Alعلى ٦٠٠مللي لتر مياه في القارورة سعتها ١٠٠٠ سم ٣ وأكمل القارورة حتى علامة ١٠٠٠ سم٣ بالماء، لتحصل على محلول شبه بتركيز ١٠٠٠٠ مجم /لتر (١%)

**خطوات التجربة**

١. أجمع ٨ لتر عينة من المياه العكرة .
٢. ضع في كل كأس من الكئوس الستة كمية مقدارها ١٠٠٠ مللى **لتر** من المياه العكرة
٣. ضع الكئوس داخل الجهاز ثم أسقط ريش التقليب في الكئوس وقلب الماء بسرعة تعادل سرعة المياه بأحواض المحطة .
٤. أضف بحرص وسريعاً بواسطة ماصة جرعة الكلور المقدمة لجميع الكئوس
٥. استخدم الماصة في إضافة جرعات شبه متزايدة بالتوالي لكئوس الاختبار الستة بحرص وبسرعة بقدر الإمكان .
٦. استمر في التقليب السريع لمدة حوالي دقيقة واحدة من بداية إضافة الشبه.
٧. قلل من سرعة التقليب لتصل إلى ٢٠–٤٠ لفة /د (٣٠) واستمر في تقليب المحلول من مدة تتراوح من ٢٠إلى ٣٠ دقيقة .
٨. لاحظ سرعة بداية تكون الندف في كل كأس وشكلها وحجمها .
٩. أوقف عملية التقليب ولاحظ ترسيب الندف بالكئوس لمدة ٥–١٠ دقيقة وعادة تشير العينة ذات الغيوم إلى عدم كفاءة الترويب ، أما ذات الترويب الجيدة فتبدو رائقة لاحتوائها على ندف مجمعة ثقيلة الوزن نسبيا يسهل ترسيه
١٠. سجل النتائج بتصنيفها إلى –ضعيفة /مقبولة/جيدة/ممتازة
١١. اسحب بواسطة سيفون على شكل حرف (S) في نفس الوقت كمية المياه التي تمثل ٧ سم العلوية من الكأس وهي حوالي نصف لتر تقريباً للكاسات الستة ، وجمعها في عدد ستة مخابير لإجراء باقي الاختبارات عليها .

وتوجد عدة عوامل هامة في تقييم نتائج اختبار تحديد الجرعات

- معدل تكوين الندف
- شكل جزيئات الندف
- شفافية المياه تحت بين الندف

- حجم هذه الندف
- كمية الندف المتكونة
- معدل ترسيب الندف
- شفافية المياه للطبقة الرائقة

وتختلف تركيزات الشبه المستخدمة في اختبار الكؤوس تبعا لاختلاف درجة عكارة المياه الخام أو بالتالي كمية الجرعة المطلوبة للمعالجة كما هو موضح بالجدول

تركيز المواد الكيماوية (الشبه) المستخدمة في اختبار (Jar Test )

| الجرعة المطلوبة ملليجم /لتر | عدد الجرامات /لتر (التحضير) | ١مللى من المضاف لكل كأس يساوى | تركيز المحلول % |
|---|---|---|---|
| ١-١٠ | ١جم/لتر | ١ملليجم/لتر | ٠.١% |
| ١٠-٥٠ | ١٠جم/لتر | ١٠ملليجم/لتر | ١% |
| ٥٠-٥٠٠ | ١٠٠جم/لتر | ١٠٠ملليجم/لتر | ١٠% |

## اختبارات التحكم في التشغيل

توجد خمسة اختبارات تساعد على التحكم في عملية الترويب/ا لتنديف وهي كالآتي:

١. اختيار تحديد الجرعات ( Jar Test)
٢. اختبار الرقم الهيدروجيني(Ph)
٣. قابلية الترشيح
٤. جهد زيتا(Zp)
٥. الشفافية (أو العكارة)

## الرقم الهيدروجيني(ph)

يجب معرفة القائم على التشغيل لدرجة التأين الهيدروجيني المثلى لكل نوع من المروبات ويعطي أفضل نتائج تساعده على اختيار المروب الملائم أو يعمل على تصحيحها و ضبطها قبل إجراء عملية الترويب ، كما أن قياس الرقم الهيدروجيني (Ph) في عينات المروق تعطي الدلائل الصحيحة على جودة الترويب .

## جهد زيتا (Zp

يستعمل اختبار جهد زيتا للمساعدة على تعيين أفضل درجة تأين هيدروجيني وأفضل جرعة للمروبات مثل الألومونيوم+++Al ، لذا يجب مراقبة جهد زيتا(Zp) ليتراوح غلباً من ٤-١٠ مللى فولت

ويبين الجدول الآتي درجة الترويب التي تحدث في نطاقات مختلفة من جهد زيتا (Zp)

**درجة الترويب في نطاقات مختلفة لجهد زيتا**

| متوسط جهد زيتا | درجة الترويب |
|---|---|
| (٣٠)-(صفر) | الحد الأقصى |
| (-١)-(-٤) | ممتازة |
| (-٥)-(-١٠) | متوسطة |
| (-١١)- (-٢٠) | ضعيفة |
| (-٢١)- (-٣٠) | معدومة تقريبا |

## اختبار قابلية الترشيح

يستخدم اختبار قابلية الترشيح كطريقة لاختبار مدى الفاعلية التي يمكن بها ترشيح مياه مروبة ، ويقيس الاختبار مقدار المياه المرشحة في مدة معينة ويجرى عادة بترشيح كمية صغيرة من مياه المحطة المروبة و المندفة فعلا (قبل الترسيب) في مرشح قطره ١٥ سم (مرشح استرشادي) ويحتوى على نفس نوع وعمق وسط الترشيح المستعمل في مرشحات المحطة ويزود هذا المرشح بجهاز لقياس العكارة ليسجل عكارة المياه الخارجية حيث تجري قياسات مستمرة لعكارة المياه المرشحة ومن مميزات هذا الأسلوب في التحكم في التشغيل ما يلي :

- أن الماء المستعمل في هذا الاختبار هو الماء المروب فعلا بالمحطة وليس لعينة مماثلة يتم تحضيرها في المعمل .
- أن الماء عند أية نقطة يستغرق نحو ١٠ دقائق للمرور خلال المرشح .

ولما كانت مدة المكث في حوض الترسيب بحجمه الكامل وفي المرشحات الفعلية تستغرق نحو ساعتين إلى ثلاث ساعات لذلك فأن التغيرات في جرعات المروب يمكن أن تجرى بناء

على أداء المرشح قبل حدوث تدهور خطير فيه ، وبصفة عامة فأن إجراء اختبارات ( Jar Test) واختبار (ph) ودرجة العكارة والشفافية (اختبارات الحكم في التشغيل) تعطى دلالة جيدة عن النتائج المتوقعة إلا أن تشغيل المحطة على الواقع قد لا يتطابق مع هذه النتائج ، لذا يجب مراقبة الأداء الفعلى من أجل :

- ملائمة الجرعة المناسبة لنوعية المياه المستخدمة .
- ملائمة الخلط السريع وحسن انتشار المادة المروبة
- ملائمة عملية التنديف البطئ

خزانات المياه المرسبة والمرشحة

ويعتمد التنديف الناجح على الخلط السريع والتام ورغم أن الترويب يحدث في أقل من ثانية واحدة إلا أن حوض الخلط يهيئ مدة مكث تصل إلى ٦٠ ثانية ويجب أن يكون الخلط عن طريق عملية الدوامات التي تحدث في حركة المياه حتى ينتشر المروب في كل حوض الترويب .

ويري البعض أنه خلال العشر ثوان الأولى من نقطة الإضافة يجب أن يخلط المروب اختلاطا تاما مع كل قطرة مياه لبدء تكوين ندف فعالة وإلا فأن كفاءة العملية كلها تهبط ويتضح تأثير معظم الاستعمال الخاطئ للمروب عند هذه النقطة ، ويمكن تصحيح هذه المشكلة أما بتعديل نقطة الإضافة أو طريقة الخلط

## ضبط طلمبةحقن الشبة

و لضبط طلمبة حقن الشبة يجب أن تتوافر المعلومات الاتية:

١. تركيز محلول الشبة ( ٢٠-٣٠% )
٢. كمية تصرف المياه الخام
٣. اقصى تصرف لطلمبة حقن الشبة
٤. الجرعة المراد إضافتها

يتم التعويض في القانون التالي لمعرفة مشوار طلمبة الشية الواجب ضبط الطلمبة عليه للحصول على الجرعة المطلوبة.

$$stroke = \frac{flowrate(m^3/hr) * dose(gm/m^3) * 100}{concentration * pump\max output * 1000}$$

و للحصول علي التركيز المطلوب نتبع الخطوات السابق ذكرها

وللحصول علي قيمة أقصى تصرف لطلمبة حقن الشبة يتم معايرة الطلمبة تبعا للخطوات الآتية

يتم تحضير محلول الشبة بنفس تركيز المستخدم بالمحطة في وعاء مدرج حجم ٥ لتر و يتم توصيل سحب الطلمبة المراد معايرتها بهذا الوعاء و يندأ تشغيل الطلمبة بعد ضبط الطلمبة علي ١٠٠% من المشوار .

- يتم حساب كمية الشبة التي تم سحبها و تكرار هذه الخطوات ل (٤٠ ,٥٠, ٦٠, ٧٠ ,٨٠ ,٩٠% ) من المشوار و رسم منحنى أداء الطلمبة و يمكن من خلال هذا المنحنى معرفة تصرف الطلمبة لكل خطوة من خطوات المشوار ..

# الفصل الثانى عشر

# الترويق

# الباب الثانى عشر

# الترويق (Clarification)

الغرض من عمليات الترويق هو العمل على ترسيب أكبر نسبة ممكنة من المواد العالقة ، والتي لها ثقل أكبر من دفع الماء ، وذلك عن طريق تركها ترسب تحت تأثير وزنها ، وعملية الترسيب إما أن تكون طبيعية ، أي تحت تأثير وزن المواد العالقة بدون إضافة أي مواد وتسمى"الترسيب الطبيعي" أو " الترسيب الذاتي" أو تكون بإضافة مواد مساعدة كيماوية للماء لتساعد على تجميع المواد الرفيعة والتي لا تهبط بمفردها في الحالة الطبيعية وتسمى "الترسيب باستعمال المروبات " ، وهو النوع الشائع في عمليات تنقية المياه وخاصة بعد زيادة المتطلبات على المياه

وهناك عوامل كثيرة تؤثر في كفاءة عمليات الترسيب منها

١. **تركيز المواد العالقة في المياه**
٢. **شكل وحجم وكثافة الواد العالقة**
٣. **درجة حرارة الماء ودرجة لزوجته**
٤. **مدة بقاء الماء في الحوض**

## نظرية الترسيب الطبيعي

يحدث الترسيب الطبيعي نتيجة هبوط الحبيبة المنفردة ، والحبيبة المنفردة (Discrete ) هي التي لا تتغير في الحجم ولا الشكل ولا الوزن ، ولا تتحد مع حبيبة أخرى أثناء عملية الهبوط في الماء ، فإذا تركنا حبيبة من الرمل مثلا ، تهبط في حوض به ماء ، نجد أنها تهبط تحت تأثير وزنها إلى أسفل ومقاومة الماء إلى أعلى ، وحيث أن قانون نيوتن يوضح أنه إذا توازنت قوى مؤثرة على حبيبة عالقة في الماء ، فلا توجد عندئذ عجلة تسارع ، وإنما سرعة هبوط ثابتة منتظمة تسمى " سرعة هبوط الحبيبة".

وتعتمد نظرية تصميم أحواض الترسيب على أن الحبيبة الداخلة ، والتي لها سرعة دخول الماء الأفقية ، ولها سرعة رأسية هي سرعة الهبوط ، يجب أن تهبط إلى القاع قبل أن تبدأ المياه في الخروج من الحوض من الجهة الأخرى ، وعلى هذا الأساس تتحدد أبعاد الحوض .

تقاوم الجسيمات غير القابلة للترسب عملية الترسيب لسببين أساسين هما :

أ – حجم الجسيمات

ب –القوى الطبيعية بين الجسيمات

## حجم الجسيمات

تحتوي المياه الطبيعية الخام على ثلاثة أنواع من جسيمات المواد الغير قابلة للترسيب وهي مدرجة من الأكبر إلى الأصغر كما هو مبين بالجدول رقم (٢-١) وهي :

أولاً : معلقة (Suspended)

ثانياً: غروية (Colloidal)

ثالثاً: مذابة (Dissolved)

**أولاً: المواد الصلبة المعلقة**

هي جسيمات ينقلها معه الفعل الطبيعي للمياه الجارية وهي أكبر من (٠.٠٠١مم) والجسيمات التي لا تتجاوز (٠.٠١مم) حجماً لا تترسب بسرعة وهي تسمى في عمليات معالجة المياه "مواد غير قابلة للترسب" أما الأكبر حجماً (أكبر من ٠.٠١مم) فتوصف بأنها قابلة للترسيب ، حيث أنها تترسب تلقائيا إلى القاع أو حوض الترسيب في غضون أربع ساعات

**ثانياً : المواد الصلبة الغروية**

هي جسيمات الطمي الدقيق والبكتريا والجسيمات المسببة للون والفيروسات وهذه الغرويات لا تترسب تلقائياً في مدة زمنية معقولة كما هو موضح بالجدول رقم (٢-١) وبالرغم من أنه لا يمكن رؤيتها بالعين المجردة إلا أ، تأثيرها قد يشاهد عند مرورها خلال عمليات المعالجة إذا لم تروب وتندف كما يجب

**ثالثا: المواد الصلبة المذابة**

هي أي مواد عضوية أو غير عضوية مثل الأملاح والكيماويات التي من أصل نباتي أو حيواني أو غازات يمكن ذوبانها في المياه ، والمادة الصلبة المذابة هي جسيمات صغيرة جداً (في حجم لجزئ) ولا يمكن رؤيتها بالعين المجردة وغالباً تكون المعادن المذابة في الماء مواد صلبة غير قابلة للترسب وتسبب مشكلات أخرى مثل الطعم أو

اللون أو الرائحة و ما لم تحول إلى مواد مترسبة بواسطة وسيلة كيمائية أو فيزيقية فلا يمكن إزالتها من الماء

ومن الأسباب الرئيسية التي تجعل الجسيمات غير القابلة للترسيب هو صغر حجمها والذي يمكن توضيحه بالمثالي التالي.

**مثال:**

من المتوقع أن يترسب جسم من رمل خشن على شكل مكعب طول ضلعه ١ مم بحيث يهبط نحو ٣. متر كل ٣ ثوان [جدول رقم (٢-١) ] والآن لنفرض أن حبيبة ٠.٠٠٠٠٠١ مم (هذا الطحن يماثل قوى التعرية الطبيعية) فإن الوزن الإجمالي لكل الجسيمات المطحونة يساوي وزن الحبيبة الأصلية للرمل الخشن .

وبمقارنة المساحة السطحية في الحالتين نجد أنها قد زادت من ١ مم٢ (وهي مساحة رأس دبوس كبير) إلأى ٦ مم٢ ، وهذه الزيادة في المساحة السطحية تسبب زيادة هائلة في قوى المقاومة والتي تقاوم الترسيب الطبيعي التلقائي وطبقاً للجدول رقم (٢-١) فبدلا من أن ترسب تلك الجسيمات الدقيقة إلى عمق (٠.٣متر) في ٣ ثوان فإنها تستغرق الآن نحو ٢٠ سنة لترسب إلى نفس العمق لذلك فمن اللازم لتعجيل الترسيب أن يتم تجميع الجسيمات الدقيقة معاً لتكون جسيمات أكبر يمكن ترسيبها بسهولة وسرعة

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القوى الطبيعية

تحمل عادة الجسيمات الموجودة في المياه شحنات كهربية سالبة وكذلك فإنه توجد هناك قوة تنافر بين أى جسمين متماثلين في الشحنة ,وفي عملية معالجة المياه تسمى قوة التنافر الكهربية جهد زيتا"ZP" وهي قادرة على إبقاء الجسيمات الغروية الصغيرة جدا متباعدة عن بعض ومعلقة في الماء.

وتوجد قوة أخرى هي قوة فان درفال بين جميع الجسيمات في الطبيعة وهي تعمل على جذب أي جسمين معاً ، وقوة الجذب هذه تعمل في اتجاه مضاد لجهد زيتا وطالما كان جهد زيتا أقوى من قوة " فان درفال" فإن هذه الجسيمات تظل معلقة .

ومن الناحية النظرية البحته فإنه يمكن توضيح عملية الترسيب –كما هو موضح في الشكل كمدخل لتفهم أسس تصميم عملية معالجة المياه ، وذلك على أساس أن المواد العالقة متجانسة التوزيع في الماء وبفرض ان سرعة المياه بما فيها من مواد عالقة في الاتجاه الأفقي "ع أ" وأن تصرف المياه يساوى" ك" وأن عرض الحوض "ض" وعمقه "ق" وتكون مساحة مقطعه "ض× ق" وتكون

سرعة المياه الأفقية = معدل التدفق /مساحة المقطع

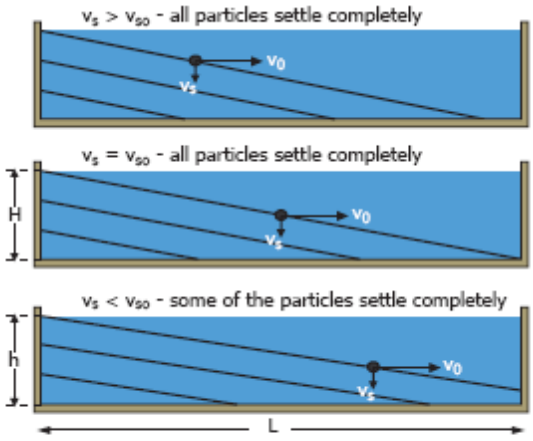



*Figure 7 - Settling of a suspension in a horizontal flow*

وتكون هي أيضا السرعة الأفقية للمواد العالقة بمختلف أحجامها

"ع أ" سرعة الأجسام العالقة في الاتجاه الأفقي

"ع هـ" سرعة الأجسام العالقة في الاتجاه الرأسي (سرعة الهبوط)

ق عمق الحوض

ض عرض الحوض

ح المساحة السطحية لحوض الترسيب

أ ء مسار المواد التي لا ترسب وتخرج من الحوض

ب جـ مسار المواد التي لا يرسب منها إلا ما كانت نقطة دخوله بالقرب من القاع

أ هـ مسار الأجسام ذات الكثافة والحجم الأكبر والتي ترسب بسرعة

أما سرعة المواد العالقة في الاتجاه الرأسي "ع أ" فتكون مختلفة حسب حجم الجزيئات وكثافتها ، ومن الشكل نستنتج أن :

أي أن :

سرعة الهبوط الرأسية للحبيبات ≤ معدل التدفق (التصرف) / المساحة السطحية لحوض الترسيب

أي أن لضمان ترسيب المواد العالقة يجب ان يكون

سرعة الترسيب ≤ معدل التحميل السطحي

**ولذلك فان معدل التحميل السطحي يجب أ،ن يكون أصغر ما يمكن لضمان جودة الترسيب ولكن يجب أن لا نغفل أهمية الزمن وأنه هو العامل الرئيسي في عملية الترسيب .**

ومن ذلك نرى أن المساحة السطحية لحوض الترسيب (ض× ل) ، لها تأثير مباشر على كفاءة الترسيب ، فكلما زادت المساحة السطحية ، قلت ""ع هـ" وزادت كفاءة الحوض (معدل التدفق) وتسمى "عهـ" بمعدل التحميل السطحي . لحوض الترسيب ، وتتخذ كأساس من أسس التصميم لهذه الأحواض ، وتكون وحدتها متر /ساعة .

ومن الشكل رقم (٢-١) يتبين أن المسار "أء" هو مسار المواد التي لها سرعة هبوط رأسية أقل من "عهـ" فيبين مسار المواد التي لها كثافة وحجم أ:بر ، وترسب بسرعة لأن سرعتها الرأسية تكون أكبر من "ع هـ" .

التطور التاريخي لعملية الترسيب:

أقدم وأبسط طرق ترسيب المواد العالقة بالماء كانت الترسيب الطبيعي (Plain Sedimentation) بأن يترك الماء ساكناً أو متحركا ببطء شديد خلال أحواض صناعية أو طبيعية إلى أن ترسب المواد العالقة أو معظمها ، ويتم سحب المياه الرائقة من السطح العلوي للحوض ، والترسيب الطبيعي لا يشمل إضافة مواد كيماوية ، وكميات الرواسب التي يمكن التخلص منها بهذا النوع من الترسيب تتراوح بين ٦٠% إلى ٧٠% مع نسبة مماثلة من البكتريا ، وفي بعض الأحوال المواتية تصل هذه النسبة إلى ٨٠%.

ويتوقف حجم خزان الترسيب الطبيعي على :

- كمية المياه المطلوب ترويقها
- حجم المواد العالقة ودرجة قابليتها للرسوب

درجة النقاوة المطلوبة

فإذا كانت المواد العالقة كبيرة فإن فترة الترسيب (Detention Period ) تكفي أن تكون بضع ساعات ، في حين أنه في حالة الحاجة للحصول على نقاوة كبيرة وكانت الجزيئات العالقة صغيرة ، فأن وقت الترسيب قد يمتد إلى أيام .إلا أن هذا النظام لم يعد مجدياً مع

التزايد المستمر في كميات المياه المطلوبة وذلك نتيجة للتزايد الكبير في تعداد السكان والتقدم الاجتماعي .

## الترسيب بمساعدة مواد الترويب (Sedimentation with Coagulants)

عندما تبين عدم ملائمة الترسيب الطبيعي للطلبات المتزايدة على المياه ، وأيضا لعدم جدواه في ترسيب الحبيبات الدقيقة والخفيفة ، وخاصة المواد الغروية ، والعوالق الطينية ، والكائنات الحية الدقيقة والمواد العضوية الناتجة من تحلل النباتات المائية ، وذلك إما لصغر سرعة الترسيب بشكل كبير وبالتالي تحتاج إلى مدة مكث كبيرة ، أو لآن الجسيمات الموجودة بالماء تحمل شحنة كهربية سالبة ، وبالتالي يحدث تنافر بينها لتماثل الشحنات ، فتبقي الجسيمات متباعدة عن بعضها ، لذلك تضاف إلى المياه مواد كيماوية (مروبات) تعمل على تجميع هذه الحبيبات الرفيعة –كما سبق توضيحه في الشكل رقم (١–٢) والذي سيتم شرحه تفصيلا في الفصل الثالث : الترويب والتنديف" وبعد عمليتي المزج السريع والمزج البطئ تمر المياه في أحواض الترويق حيث ترسب الندف المتكونة في أحواض الترويق بما جذبت إلى سطحها من مواد عالقة إلي قاع الحوض .

ولا تختلف أحواض الترويق في تصميمها عن أحواض الترسيب الطبيعي كما أن العوامل المؤثرة على كفاءة الترسيب فيها لا تختلف عن أحواض الترسيب الطبيعي ولكن في وقت أقل جدا ، وبالتالي تكون أحجام هذه الأحواض أصغر كثيرا ، وبالرغم من اختلاف أنواع أحواض الترويق (الترسيب بالمروبات ) . إلا أنها تتفق في الأسس الرئيسية للتصميم وإن اختلفت في بعض التفاصيل .

وقد تطورت هذه الأحواض بالتقدم العلمي المستمر ، وتزايد متطلبات المياه واتجه هذا التطور في اتجاهين : الأول هو زيادة سرعة الترسيب ، و الثاني هو زيادة سرعة مرور المياه في الأحواض ، وذلك لتقليل حجم الأحواض قدر الإمكان كما في الحوض النابض (Pulsator) .

ثم اتجهت الأبحاث إلى تعويم الندف بدلا من ترسيبها بطريقة التعويم بالهواء (Air Floatation) ، وطالما وجد العلم والعلماء فلن تتوقف المحاولات لتحسين الأداء .

## أنواع أحواض الترسيب

تختلف أنواع وأشكال الترسيب (Sedimentation Basins ) باختلاف التصميمات فمنها

## أ – الأحواض المستديرة :

ذات التغذية الرأسية ويكون سريان الماء في اتجاه رأسي ويسمى ذلك بالتصريف الرأسي ، ويوضح حوض ترسيب دائري ذو تصرف رأسي .

## ب –الأحواض المستطيلة

وفيها يكون سريان الماء في اتجاه واحد موازي لطول الحوض ويسمى ذلك بالتصريف في خطوط مستقيمة .

وفي كلا النوعين يلزم الحفاظ على سرعة المياه وتوزيع تصرف المياه منتظمين قدر الإمكان لمنع تكون الدوامات والتيارات الدوامية بها والتي تعوق ترسيب المواد العالقة ، مع تفادي حدوث قصر الدورة (Short Circuit) بين المدخل والمخرج حيث ينقسم حوض الترسيب عادة إلى عدة مناطق أساسية هي :

**١. منطقة دخول المياه :**

ويشترط أن يكون دخول الماء الوارد من حوض التنديف منتظما وينتشر في أرجاء حوض الترسيب . ولذلك يكون دخول المياه من خلال جدار ذو فتحات متماثلة لتوزيع المياه على كل إرجاء الحوض

**٢. منطقة الترسيب :**

وهي أكبر مناطق الحوض حيث يتم ترسيب العوالق بمكوثها مدة كافية من الزمن ، ولكون المياه تسير بسرعة أكبر نسبياً في هذه المنطقة ، فيلزم عمل الاحتياطات لتفادي التيارات الاعصارية .

**٣. منطقة تجميع الروبة:**

وهي تقع عادة في قاع الحوض ، وتعتبر مكانا للتجميع المؤقت للأجسام المترسبة ، ويراعي في تصميم مدخل الماء ألا يسبب في تيارات دوامية قرب منطقة تجميع الروبة ، تتسبب في تهيج الجسيمات المترسبة فتصعد للماء الرائق مرة ثانية .

**٤. منطقة خروج الماء**

ويجب أن يوفر مخرج الماء انتقالا سلسا من حوض الترسيب إلى مجرى خروج الماء الرائق ، كما أنه يساعد في التحكم في منسوب الماء بالحوض ، وقد تستخدم حواجز بها فتحات (هدارات) على شكل حرف (V) لتوفر الخروج الهادئ للمياه الرائقة بحيث لا تحمل معها ندفا تنتقل إلى المرشحات , و لكمية المياه المارة علي الهدارمعدل يجب الا يتجاوزة لأن سرعة خروج المياه سوف تدفع الندف لتتجاوز الهدار وتخرج الي المرشحات والمعدل التحميل علي الهدار يجب ألا يتجاوز ٣٠٠ م$^{٣}$ /م/يوم .

و قي حالة زيادة التحميل علي المروق يمكن اضافة هدارات اخري وتغيير شكل خروج المياه من المروق للحصول علي معدل التحميل المطلوب .

## العوامل المؤثرة في عملية الترسيب

**تتأثر عملية الترسيب بعدة عوامل أهمها :**

حجم الحبيبات : وطريقة توزيعها ، فكلما زاد حجمها ووزنها ، ازدادت كفاءة الترسيب

شكل الحبيبات : فكلما اقترب شكلها من الشكل كلم كان ترسيبها أسرع وأكفأ .

كثافة الحبيبات : فكلما زادت كتلتها بالنسبة لحجمها وزادت سرعة رسويها وبالتالي كفاءة الترسيب

درجة حرارة الماء : فكلما ارتفعت درجة حرارته قلت كثافته ولزوجته وبالتالي زادت سرعة رسوب الحبيبات وزادت كفاءة الترسيب

الشحنة الكهربائية للجسيمات : والتي تكون دائما سالبة الشحنة ، وعند معالجة المياه بالشبه موجبه الشحنة ، يحدث تجاذب بين الجسيمات السالبة والموجبة مما يساعد على ترسيبها وزيادة كفاءة الترسيب .

سرعة سريان الماء في الحوض : فكلما قلت سرعة الماء ، زادت كفاءة الترسيب ، ويفضل إلا تتجاوز السرعة الأفقية في الحوض ( ٣٠سم/ دقيقة) .

مدة بقاء الماء في الحوض (مدة المكث –Detention time ) : فكلما زادت المد ة، زادت جودة الترسيب ، ومن النواحي الاقتصادية والعلمية أن تكون مدة المكث في حدود ٣–٤ ساعات ، حيث أن زيادة المدة أكثر من اللازم ، لا يزيد من كفاءة إلا بنسبة بسيطة .

حجم حوض الترسيب

مدة المكث = معدل التصرف خلال الحوض

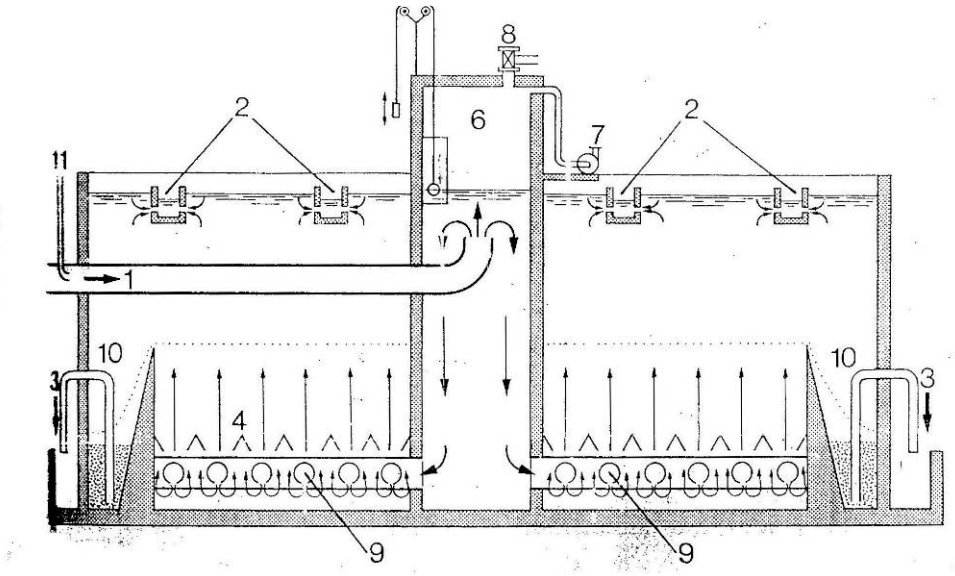
النسبة بين طول وعرض حوض الترسيب في الأحواض المستطيلة وذلك لإقلال فرص تكون مناطق راكدة أو ميتة (Dead Zones) عند زيادة عرض الحوض.

## الترويق باستخدام بطانية الروبة

تختلف هذه المروقات عن مروقات الترسيب الطبيعي من حيث الحجم حيث تسمي هذه المروقات ذات الحمال العالية حيث تصل معدل التحميل السطحي بها الي ٣م/س .

فكرة العمل :

و الفكرة الأساسية لهذا النوع من المروقات هي تكوين بطانية من الروبة تطفو قريبا من السطح يمر الماء المندف من خلالها ليتم حجز الندف خلال هذه الطبقة و يخرج الماء المروق نقيا الي المرشحات .و طريقة تكوين هذه الطبقة العائمة من الندف المتجمعة تختلف من مروق إلى الآخر ولكن تشترك جميعها في أن المياه المروبة المحماة بالندف تصطدم بقاع المروق وترتد لتطفو الى العلى ولكن نظرا لثقل وكثافة الندف لا تخرج مع المياه المروقة بل تبقى معلقة على ارتفاع ٠.٩ م من القاع لتكون بطانية الروبة .




ومن هذه المروقات ( المروق النابض ) pulsator وهو من انتاج شركة ديجرمون الفرنسية و هو المنتشر في محطات القاهرة الكبرى وسائر الجمهورية ,

## نظرية عمل المروق النابض

بتم تكوين بطانية الروبة من خلال دخول المياه الي غرفة تفريغ تتوسط المروق ويتصل بها مجموعة من المواسير التي تمتد بطول المروق , ومن خلال هذه الغرفة يتم تفريغ الهواء لتمتلئ بالمياه العكر المختلط بالشبة وعند ارتفاع المياه إلى منسوب معين يتم تفريغ المياه عن طريق إسقاط عمود المياه خلال زمن صغير فتتكون النبضة و يخرج الماء مندفعا إلى المروق من خلال المواسير المثقبة لأسفل فيصطدم الماء بقاع المروق و تتجه الندف إلى أعلى و تبقى معلقة بفعل وزنها وتكون بطانية الروبة وتسمى هذه العملية بالنبضة .

يتحكم في تفريغ الهواء والماء من المروق طلمبة تفريغ و محبس دخول الهواء الذي يتم ضبطه وفقا للتصرف الوارد وكمية الشبة المضافة و عكارة المياه الخام .

تتميز هذه المروقات النابضة بجودة المياه المروقة عند ضبط التشغيل , وكذلك صغر المساحة وإمكان التطوير .

**من عيوب هذا النوع من المروقات**

صعوبة الضبط و الحاجة للمراقبة الدائمة والصيانة بالمعدات الميكانيكية .

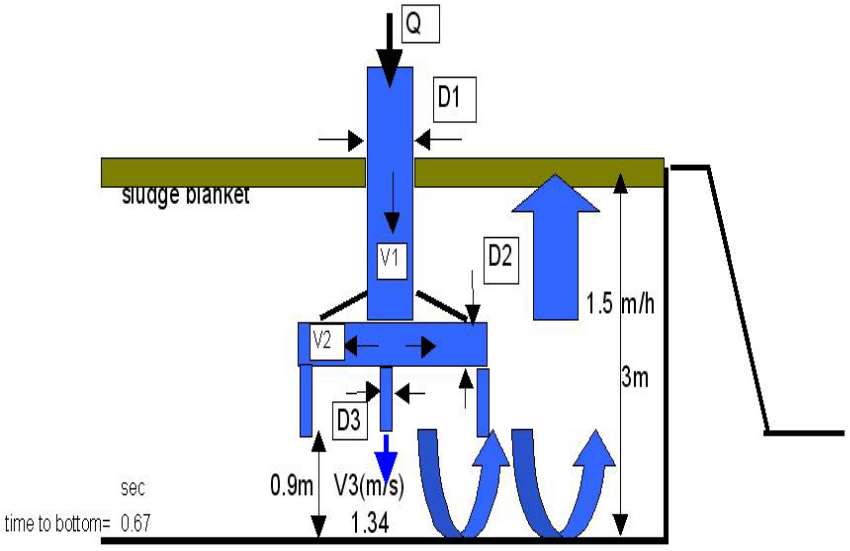
و أيضا من أنواع مروقات بطانية الروبة

**مروقات الترايدنت : وهي من إنتاج شركة BWT الانجليزية**

الكرة الأساسية لعمل المروق : تعتمد فكرة تشغيل المروق علي تكوين بطانية الروبة المعلقة نتيجة اصطدام الماء المروب بقاع المروق وصعود الندف لأعلى لتكون بطانية الروبة .

يختلف هذا النوع من المروقات عن المروق النابض أن دخول الماء لا يعتمد على طريقة ميكانيكية بل يدخل الماء بسرعة إلى المروق نتيجة فرق مناسيب هيدروليكيا وهو ما يعني عدم الحاجة إلى الصيانة والضبط المستمر لضمان تكون بطانية الروبة .

تدخل المياه إلى المروق عن طريق أنابيب ذات ثلاثة فروع بها ثقوب صغيرة وهذه الأنابيب تمتد من أعلى المروق إلى أسفل حيث تخرج المياه بسرعة لتصطدم بالأرض ودون أن تتكسر الندف و تكون بطانية الندف .



## التخلص من الروبة

تتكون الروبة من مخلفات المادة المروبة (السبه) مع رواسب صلبة ، وتشغل الروبة ٣-١٠% من حيز الحوض ، وفي أحواض الترسيب ذات التدفق الأفقي يتم ترسيب ٥٠% من الندف في الثلث الأول من طول الحوض ، ويراعي ذلك في تصميم نظام استخراج الروبة من الحوض .

ويجب استخراج الروبة من الحوض دورياً للأسباب الآتية :

أ - منع تداخلها مع المياه المروقة وإعادة تعكيرها .

ب - منع تكاثر البكتريا التي تسبب طعما ورائحة غير مقبولين للماء

ت - تفادي شغل حيز كبير من الحوض ، يخفض من كفاءة تشغيله مع انخفاض نوعية المياه المنتجة لإنخفاض مدة المكث

ويتم استخراج الروبة من الأحواض إما يدويا أو ميكانيا عن طريق

١) الكاسحات الميكانيكية للأحواض المستديرة .

٢) الكراكات الأفقية والكباري الكاسحة للأحواض المستطيلة .

٣) حجرة تركيز الرواسب .

## أنظمة جمع الروبة

وللتخلص من الروبة من الأحواض الصغيرة يتم ذلك يدويا أما في المروقات الكبيرة التقليدية فيكون هناك منظومة للتخلص من الروبة ففي الأحواض الدائرية تكون هناك كساحة لجمع الروبة الترسبة من كل المروق الي مكان معين حيث يتم سحبها الي بيارة ترسيب قبل التخلص منها نهائياً .

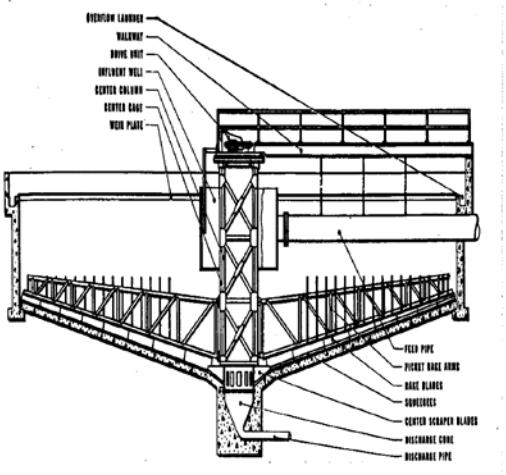
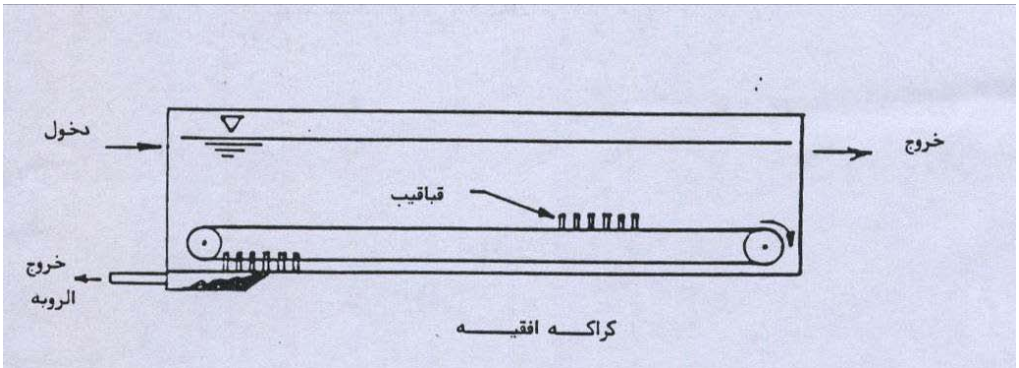
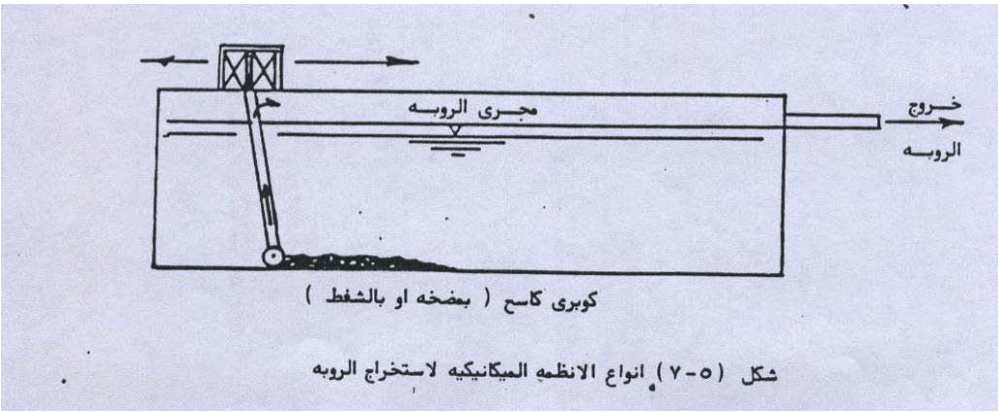



Fig. 5-19 Continuous flow thickeners.

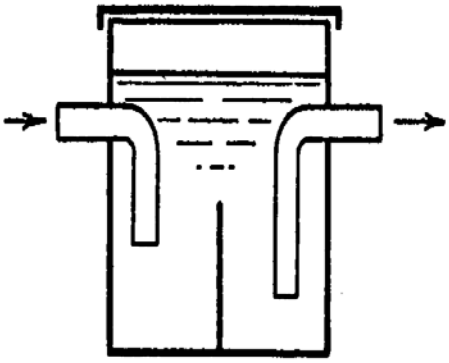


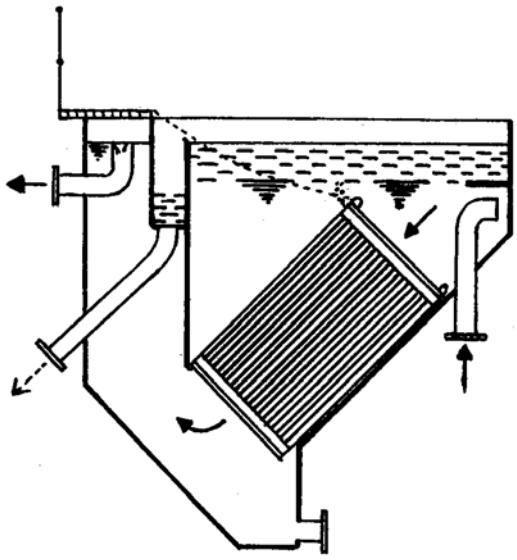

شكل (٥-٧) انواع الانظمه الميكانيكيه لاستخراج الروبه

يتم بعد ذلك تجميع الروبة في بيارات معالجة الروبة والتي تعمل علي ترسيب اكبر قدر من المواد العالقة ثم التخلص من المياه الناتجة في المصارف علي ان تكون مطابقة للقوانين البيئية

هذه البيارات لها العديد من الأشكال



و من تلك الأنواع أحواض الترسيب باستخدام ألواح اللاميلا .



و قد تستخدم بحيرات الترسيب الطبيعي لمعالجة مياه صرف المروقات وغسيل المرشحات كما في محطة كدوان بالمنيا.

## تطوير مرحلة الترسيب

نظرا للزيادة السكانية وزيادة الاستهلاك المياه ظهرت الحاجة إلي زيادة كميات المياه المنتجة و اضطر السادة العاملين بالمحطات إلي التحميل علي المحطة لإنتاج اكبر كمية ممكنة من المياه في حدود المواصفات ولكن لتغير خواص المياه الخام نتيجة الصرف الزراعي والصحي صارت عملية التحميل محفوفة بالمخاطر و هذا ما دفع العاملين في مجال المياه إلي التفكير في كيفية تطوير أعمال المعالجة , و نظرا لأهمية عملية الترسيب كانت أهم المجالات التي اتجهت إليها محاولات التطوير .

و كانت ألواح اللاميلا و من بعد ذلك أنابيب الترسيب هي أهم الطرق المستخدمة في التطوير , نظرا لسهولة التطبيق و جودة النتائج .

## مميزات أنابيب الترسيب:

أ – يمكن لأنابيب الترسيب مضاعفة معدل السريات علي حوض الترسيب من ٢ الي ٤ مرات

ب –يمكن لها تقليل كمية المواد الكيماوية المضافة الي النصف

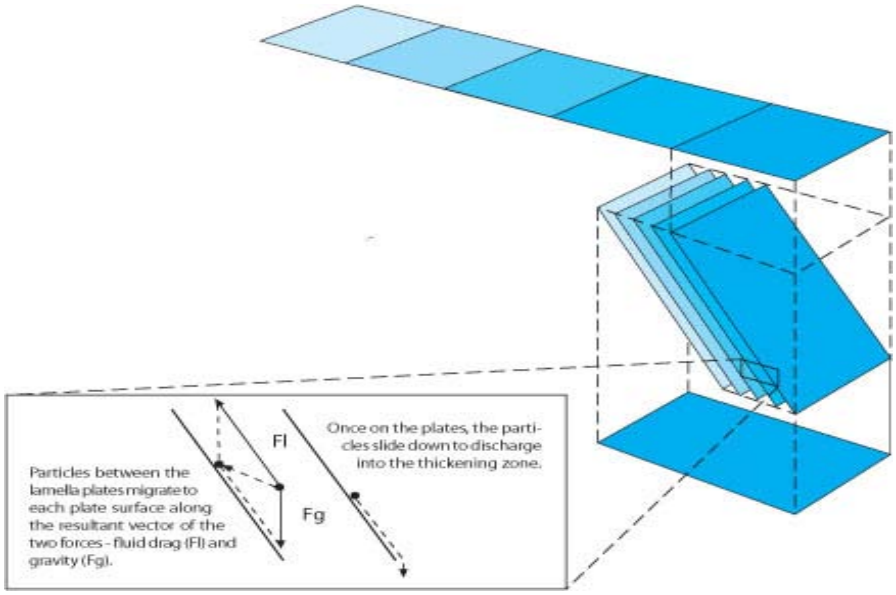
ت –تقلل الأحمال علي المرشحات وتزيد عمر المرشح

ث –تقلل كميات الماء المستخدمة في غسيل المرشحات

ج – الأحواض المصممة لتستخدم أنابيب الترسيب اصغر حجما من نظيرتها بدون أنابيب الترسيب

## نظرية العمل

تعمل اللاميلا علي أساس استخدام ألواح مائلة متعددة لزيادة المساحة المستخدمة في الترسيب حيث تكون المساحة المستخدمة للترسيب هي مجموع المساحات المسقطة علي المستوي الأفقي للألواحوكما هو مبين بالشكل.




ونظرا لصغر المسافة الراسية اللازمة للترسيب والتي تحددها المسافات بين الألواح ونظرا لكون القوي المؤثرة علي الجزيئات العالقة هي الوزن لأسفل والسريان في اتجاه ميل الألواح تكون المحصلة قوه تعمل علي ترسيب الجزيئات علي سطح ألواح اللاميلا وعند ترسبها علي سطح الألواح يصبح القوة المؤثرة عليها هي الوزن لأسفل مما يعمل علي انزلاقها إلي قاع حوض الترسيب

## تطوير المروقات التقليدية ( المروقات المستطيلة ذات السريان الأفقي )

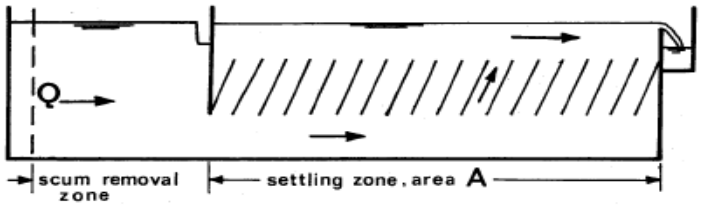



Fig. 3-36 Tilted plate separator.

يتم تطوير المروقات ذات السريان الأفقي بتركيب ألواح اللاميلا بحيث يمر الماء بين الألواح قيل الخروج من الهدار وهو ما يزيد من كفاءة الترسيب ويقلل الحمل السطحي

## تطوير المروقات الدائرية

يمكن ايضا ان يتم تطوير المروقات الدائرية بإضافة ألواح اللاميلا كما هو موضح بالشكل .

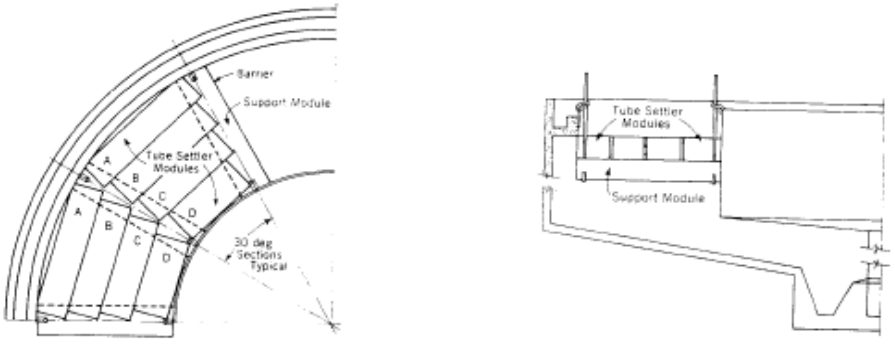


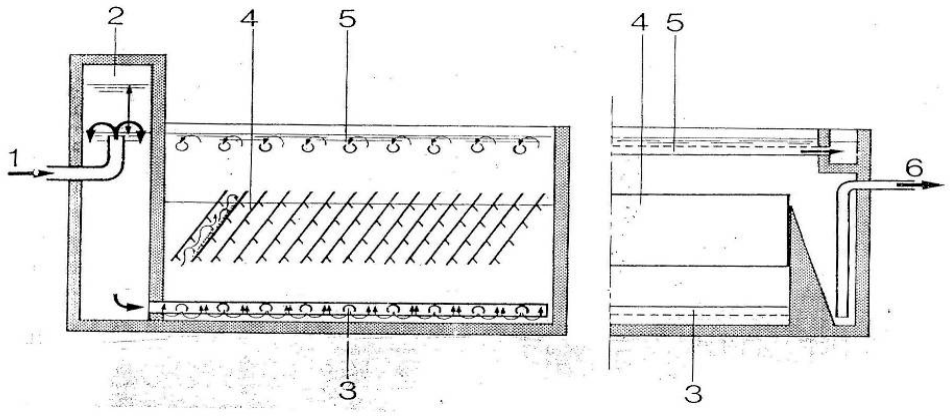

Fig. 3-41c Circular tanks with tube settling.

تطوير المروق الدائري

Installation of a Svedala LTC Combi lamella clarifier next to a conventional round thickener with approximately the same clarification area.

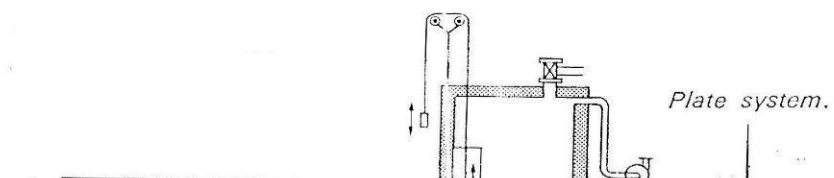
## تطوير المروق النابض باستخدام ألواح اللاميلا

١. يمكن إضافة ألواح اللاميلا في طبقة بطانية الروبة المعلقة واعلي مواسير التغذية بزاوية ٥٥ درجة علي الأفقي بحيث لا يزيد ارتفاعها عن المركز إلا بمسافة صغيرة علي أن تكون المسافة بين الألواح ١٣ بوصة طبقا لتصميمات الشركة المصممة للمروق

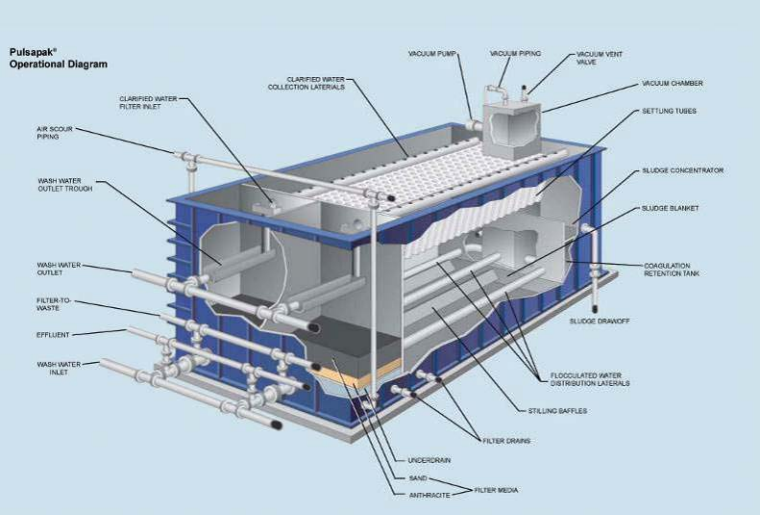



٢. يمكن إضافة ألواح اللاميلا ذاتها اعلي طبقة بطانية الروبة المعلقة بزاوية (٥٥- - ٦٠ ) درجة علي الأفقي وأسفل مستوي هدار خروج المياه المروقة ( plate pulsator ) لتقوم الألواح بتجميع الندف الهاربة وإعادتها إلى طبقة الروبة لتحسين نوعية المياه المروقة و زيادة تماسك طبقة الروبة.

٣. وكما يمكن تطوير المروق النابض باستخدام أنابيب الترسيب كما بالشكل



بمكن إضافة أنابيب الترسيب بدلا من الألواح اعلي طبقة الروبة المعلقة وأسفل هدار خروج المياه المروقة لتؤدي وظيفة ألواح اللاميلا .



## مخططات التطوير أحد أهم دعائم نجاح عمليات المعالجة:

وبعد استعراض الطرق المختلفة للتطوير يجب ان ننوه ان التطوير عملية هامة ولذا يجب ان يسلقها مراحل من الدراسة النظرية والتجارب العملية من خلال محطات تجريبية داخل المحطة المراد تطويرها لتجنب الفشل و إهدار الأموال .

المحطة التجريبية بمحطة دمنهور ٢

# الفصل الثالث عشر

# الترشيح

# الباب الثالث عشر

# الترشيح ( Filtration )

مرشحات رملية سريعة

عملية الترشيح هي عملية سريان الماء خلال طبقة منفذة سواء كانت هذه الطبقة ورقة ترشيح او وسط مسامي, وفي محطات تنقية المياه الترشيح هو عملية سريان المياه من خلال وسط حبيبي ( عادة رمل ) حيث يتم حجز الشوائب من المواد العالقة وازالة الكائنات الدقيقة الممرضة

## آلية الترشيح

عند مرور المياه خلال الوسط الترشيحي يتم حجز الشوائب ذات الحجم الاكبر من المسام الموجودة بسطح الوسط الترشيحي كما بالشكل ١ وهذا لا يحدث بالمرشحات الرملية السريعة لان هذه الشوائب عادة يكون قد تم إزالتها خلال مرحلة الترويق ,

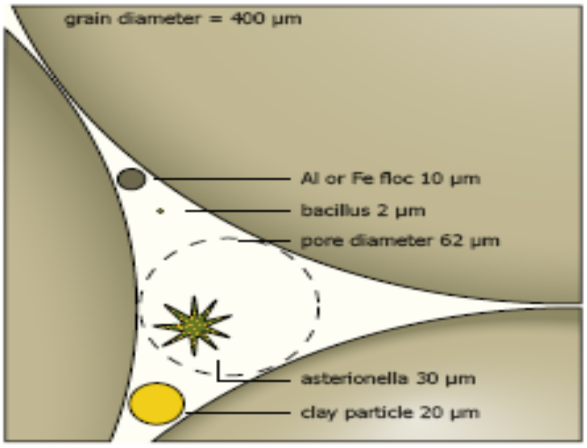



*Figure 2 - Principle of screening*

وفي حالة استخدام وسط ترشيحي ذو حبيبات صغيرة تكون ايضا المسام بين الحبيبات صغيرة وتتم عملية تصفية ( حجز الشوائب ) بما يسمي cake filtration وحيث يتم حجز الشوائب و المواد العالقة في الطبقة العليا من الوسط الترشيحي . و من عيوب هذا النوع من الترشيح انه سرعان ما يتم انسداد المرشح وتقصر فترة تشغيله.

في المرشحات الملية السريعة تتم عملية ازالة المواد العالقة والغروية داخل الوسط الترشيحي وبالتالي لكي تحدث عملية الانسداد لابد أن تنسد كل مسام الوسط الترشيحي .

تنتقل المواد العالقة والغروية الي مسام الوسط الترشيحي بعدة طرق , حيث تسري المواد العالقة والغروية مع الماء وتتبع خطوط سريانه خلال الوسط تلك الخطوط التي تماثل التركيب المعقد للفراغات بين الحبيبات ( المسام ) , وعندما تقترب المياه من الحبيبات تعترض الحبيبات طريق الجزيئات العالقة و تترسب الحبيبات الثقيلة بينما تنتشر الجزيئات الصغيرة علي سطح الحبيبات وتصطدم الجزيئات السريعة بحبيبات الوسط الترشيحي وتلتصق بها و قد يحدث تيارات درامية داخل الوسط الترشيحي تتسبب في حجز الجزيئات الصغيرة داخل الوسط الترشيحي .

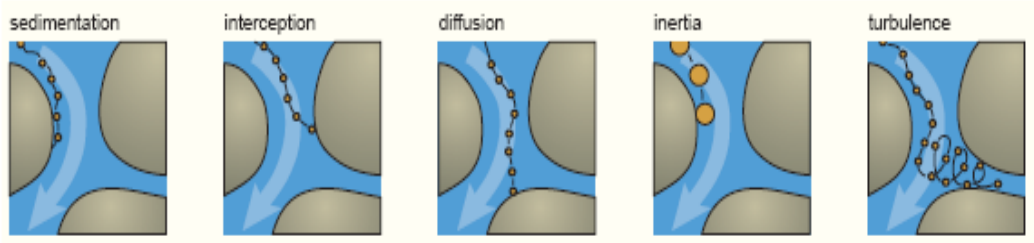



*Figure 3 - Transport of impurities towards the grain*

وبالإضافة الي العملية الفيزيائية التي تم شرحها فانه هناك أيضا عمليات كيميائية وبيولوجية تحدث اثناء الترشيح ففي ترشيح المياه الجوفية يتم إزالة الحديد و المنجنيز بواسطة الأكسجين الذي يتم إضافته في الخطوة السابقة فتتكون ندف هيدروكسيد الحديديك التي يتم إزالتها بالطرق السابق شرحها , وكذلك الحال في إزالة المنجنيز و بالنسبة الي إزالة الامونيا فان نمو البكتريا علي سطح الحبيبات هو احد العوامل الهامة لإزالة الأمونيا و المواد العضوية .

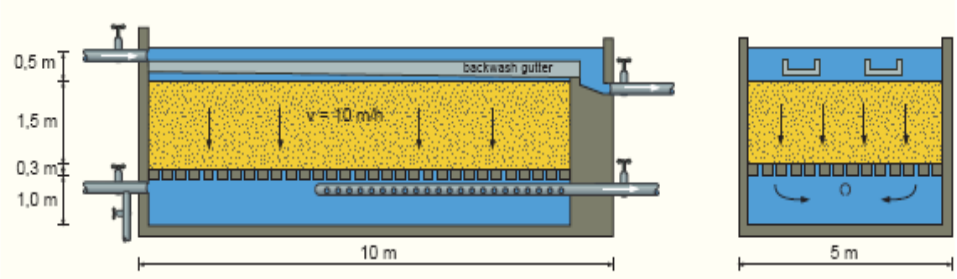



*Figure 1 - Principle of rapid sand filtration*

## كيف يعمل المرشح

عند مرور المياه داخل الوسط الترشيحي يتم حجز الجزيئات كما سبق الشرح وتدريجيا تزداد مقاومة الوسط الترشيحي للمياه ويقل التصرف الخارج من المرشح ويتم اخراج المرشح من الخدمة وغسيل المرشح بواسطة إمرار المياه في الاتجاه العكسي و تتم هذه العملية بصورة دورية لمنع زيادة المقاومة عن حد معين .

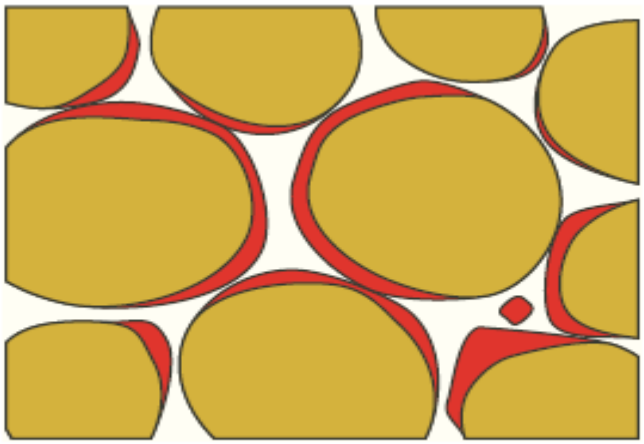


*Figure 6 - Reduction of pore volume as a result of accumulated solids*

وللتعرف علي عملية الترشيح و تخيل ما يحدث أثناء الترشيح يمكن رسم منحني الفقد في الضغط مع الزمن وهو ما يسمي (Lindquist diagram )

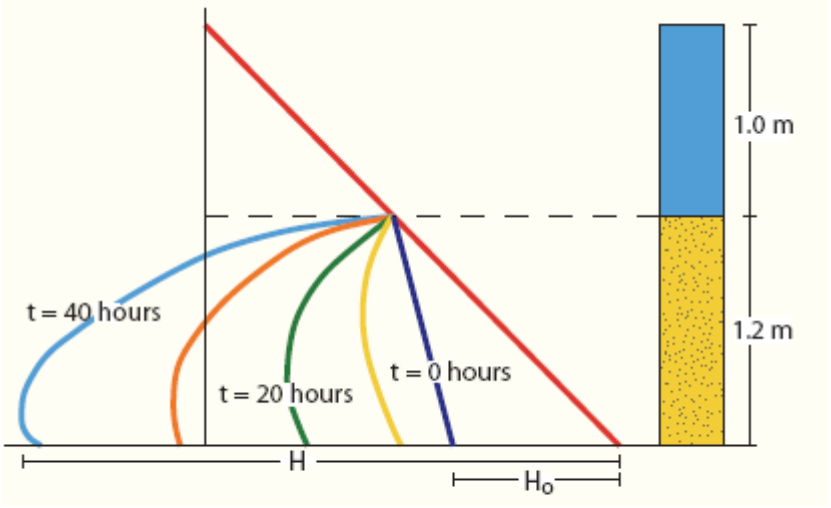



*Figure 7 - Progress of the filter bed resistance in time, the so-called Lindquist diagram*

و من خلال الرسم المنحني يمكن ملاحظة انه مع مرور الوقت وانسداد المرشح يزداد الفقد في الضغط ( و معناه الفقد في الضغط الأستاتيكي للمياه نتيجة مرور المياه من خلال مسام الوسط الترشيحي )الي ان يصل الفقد في الضغط الي القيم السالبة وهو ما يعني وجود فقاعات هواء داخل المرشح و ينخفض تصرف المرشح بشكل كبير و لذلك يجب ألا يصل الضغط الي القيم السالبة .

ويجب ملا حظة انه عند بدء الترشيح توجد قيمة للفقد في الضغط تختلف باختلاف الوسط ترشيحي و معدل الترشيح ( كمية المياه المارة م٣/س / مساحة المرشح السطحية م٢) وتتناسب مع مسامية الوسط الترشيحي .

ومن اهم المنحنيات الخاصة بالمرشح هي منحنى عكارة المياه المرشحة مع الزمن

# العوامل المؤثرة علي عملية الترشيح

## أولاً: الوسط الترشيحي

تستخدم الرمال عادة كوسط ترشيحي وذلك لتوفرها ورخص ثمنها و قد يستخدم ايضا مع الرمال مواد أخري كوسط ترشيحي مزدوج مثل الانثراسيت , وقد يستخدم الزلط ايضا كوسط نرشيحي يسبق المرشح الرملي في المرشحات الثنائية , ويستخدم ايضا كسر الجرانيت كوسط ترشيحي .

## مواصفات الوسط الترشيحي

حجم الحبيبات وتدرجها و المقصود بتدرج الحبيبات أن العينه المأخوذة من الرمل تحتوي علي حبيبات ذات أقطار مختلفة و للتعرف علي اهمية التعرف علي توزيع احجام الحبيبات بالوسط الترشيحي يمكن ان نتخيل وسط ترشيحي ذو تدرج كبير بين الحبيبات عند غسيل الوسط الترشيحي ترتفع الحبيبات الصغيرة الي اعلي ة تسقط الحبيبات الكبيرة الي اسفل وعند بدء الترشيح مرة اخري نجد ان الطبقة العليا من المرشح ذات كفاءة ترشيح اعلي بينما لا تؤثر الطبقات السفلي علي جودة المياه المرشحة مما يعجل بانسداد المرشح وقصر فترة تشغيل المرشح .

و للتعرف علي تدرج الحبيبات وحجمها يتم استخدام مجموعة من المناخل توضع فوق بعضها ويتم إمرار وزن معين من الرمال وتحديد وزن الرمال المحجوزة علي كل منخل لتحديد النسبة المئوية للأقطار المختلفة

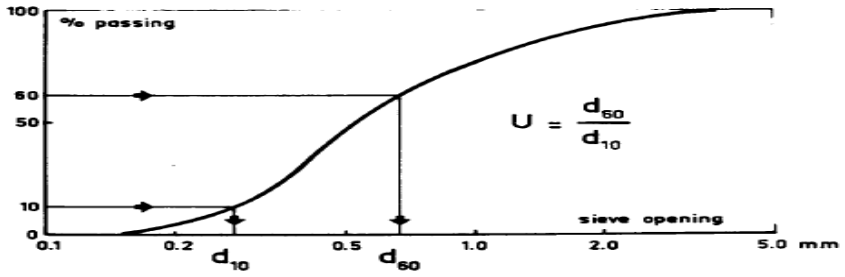



Fig. 2.23 Grain size analysis

ويتم رسم المنحني وتحديد الحجم الفعال ومعامل الانتظام حيث ان:

**الحجم الفعال**

هو مقياس فتحة المنخل التي تمرر ١٠% من وزن العينة او هو مقاس المنخل الذي يحجز ٩٠% من وزن العينة يعتمد علي معرفة الحجم الفعال في الحسابات الخاصة بالترشيح .

**معامل الانتظام**

ويعرف بأنه هو نسبة مقاس المنخل الذي يمرر ٦٠% من وزن العينة الي مقاس المنخل الذي يمرر ١٠% من وزن العينة

$$U = \frac{d_{60}}{d_{10}}$$

**المسامية :**

وهي نسبة حجم الهواء داخل الوسط الترشيحي الي الحجم الكلي للعينة و( مسامية الرمل ذات الحجم الفعال ٠.٨ مم = ٤٠% تقريبا وتختلف تبعا لمعامل الانتظام )وهي احد الخواص الهامة التي تؤثر علي معدل الترشيح و جودة المياه المنتجة وكفاءة المرشح و كذلك علي إجراءات الغسيل .

**الكثافة :**

وهي وزن حجم ١ م٣ من العينة مقاسا بالكيلوجرام .

**معدل الترشيح**

وهي = كمية المياه التي تدخل المرشح (م٣/س ) / مساحة المرشح السطحية م٢

و يختلف معدل الترشيح تبعا لنوع الوسط الترشيحي و نوع المرشح ففي المرشحات الرملية البطيئة يكون معدل الترشيح ٠.١ م/ ساعة بينما في المرشحات الرملية السريعة يكون معدل الترشيح في حدود ٤: ٦ م/ ساعة ويمكن زيادة معدل الترشيح الي ٨ م/ ساعة في حالة جودة المياه المروقة .وقد تصل سرعة الترشيح في مرشحات الترشيح من اسفل الي اعلي الي ١٨ م/ ساعة وقد تزيد علما بأنها قد تستخدم كمعالجة ابتدائية للمياه .

**جودة المياه الخام**

والمقصود هي المياه الداخلة الي المرشح حيث تؤثر جودة المياه الداخلة علي سرعة انسداد المرشح وجودة المياه المرشحة . بالنسبة للمرشحات الرملية السريعة ذات الوسط الرشيحي ذو الحجم الفعال ٠.٨ مم يجب ان يسبق المرشح عمليتي تنديف وترسيب حتي يتم التخلص من معظم المواد العالقة والشوائب للحصول علي فترة تشغيل اقتصادية للمرشح . بينما في محطات الترشيح المباشر لا توجد عملية الترسيب تسبق الترشيح ولذا يجب الا تزيد عكارة المياه الداخلة الي المرشح عن ٥ وحدات عكارة للحصول علي فترة تشغيل اقتصادية للمرشح

**جرعة المادة المروبة المستخدمة**

وهي من اهم العوامل التي تؤثر علي عملية الترشيح اذ ان الترويب والتنديف الجيد والجرعة المناسبة هي العصا السحرية لعملية المعالجة واذ تعمل الشبة علي تكوين ندف ذات حجم مناسب يمكن ترسيبها بسهولة وبالتالي تتحسن جودة المياه المروقة . وفي محطات الترشيح المباشر تكون جرعة الشبة هامة جدا اذ ان جرعة الشبة الكبيرة تقوم بتكوين ندف كبيرة مما يؤدي الي انسداد المرشح بينما جرعة الشبة المناسبة لمحطات الترشيح المباشر كما ذكرت الكتب المتخصصة والدارسين يفضل ان تكون في حدود ٢٠ جم٨م٣ .

**ارتفاع الوسط الترشيحي:**

عمق المرشح هو المكان الذي يتم فيه حجز الشوائب وهو احد العوامل المؤثرة علي جودة الترشيح اذ أن مع زيادة عمق المرشح تزداد الفترة التي يتم بها حجز الشوائب وبالتالي تزداد جودة المياه المرشحة .

و من خلال خبرات العملين يمكن القول انه توجد علاقة بين الحجم الفعال للوسط الترشيحي وارتفاع الوسط الترشيحي حيث مع صغر الحجم الفعال يفضل ان يكون المرشح ذو ارتفاع صغير وبالعكس مع وسط ترشيحي ذو حجم فعال كبير يفضل ان يكون ارتفاع الوسط الترشيحي كبيرا للحصول علي جودة عالية للمياه المرشحة وفي نفس الوقت فترة تشغيل اقتصادية للمرشح . ويمكن القول ان العلاقةبين ارتفاع الوسط الترشيحي والحجم الفعال هي

$$L/d = 1000$$

**إجراءات الغسيل العكسي**

ان الغسيل الجيد هو العامل الأهم في تحديد عمر المرشح وجودة المياه المنتجة اذ ان بدون الغسيل الجيد تتراكم الشوائب داخل الوسط الترشيحي وتقصر مدة تشغيل المرشح وتتزايد في المرشح البكتريا والطحالب وتسوء نتائج المياه المرشحة نظرا لحدوث تشققات بالوسط الترشيحي يمر من خلالها الماء بدون ان يحدث الترشيح بل قد يلزم تغيير الوسط الترشيحي بأكمله لوجود تكلسات من الطين والطمي يصعب إخراجها حتى بالغسيل الجيد .

# التطبيقات المختلفة للترشيح

## المرشح متعدد الطبقات

هو المرشح الذي يتكون من عديد من طبقات الوسط الترشيحي المختلفة من حيث حجم الحبيبات فتكون الحبيبات ذات الحجم الأكبر في الأعلى , وفي حالة الترشيح من أسفل الي اعلي تكون الحبيبات ذات الحجم الأكبر بالأسفل بحيث يمر الماء علي الوسط الخشن اولا ثم الناعم .

و في حالة استخدام أكثر من وسط ترشيحي ( الوسط الترشيحي المزدوج )– كالرمل و الانثراسيت مثلا – تنتظم الطبقات المختلفة تبعا للحجم والكثافة فتكون الحبيبات ذات الكثافة الأعلى أسفل الحبيبات ذات الكثافة الأقل و بذلك تكون الحبيبات ذات سرعة الترسيب الأعلى أسفل الحبيبات ذات يرعه الترسيب الأقل وذلك لضمان ان يبقي الوسط الترشيحي منتظما بعد الغسيل العكسي حيث تعود الحبيبات الكبر كثافة الى مكانها الطبيعي في الأسفل.

يمتاز هذا النوع من المرشحات بقدرته على حجز المواد العالقة ذات الحجم الكبير في الطبقة العليا للمرشح ويتم حجز الشوائب الأصغر بالطبقات السفلى و بذلك يتم توزيع الشوائب علي كل الوسط الترشيحي وبالتالي تقل سرعة انسداد الوسط الترشيحي مما يعني فترة أطول لتشغيل المرشح .

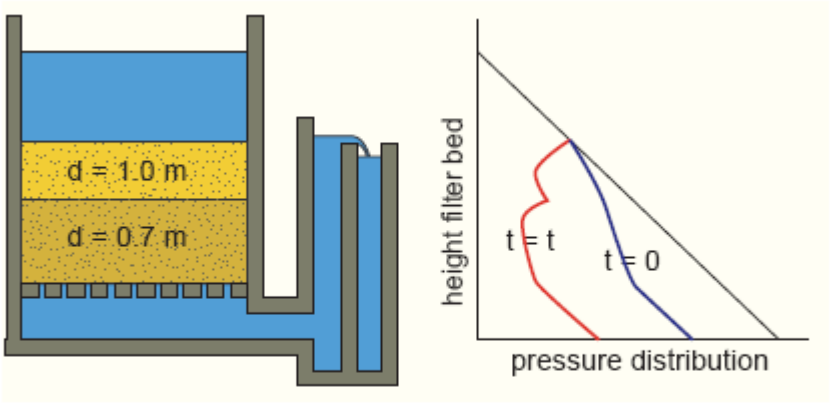



*Figure 23 - Resistance build-up for a multiple layer filter*

## الترشيح بالضغط

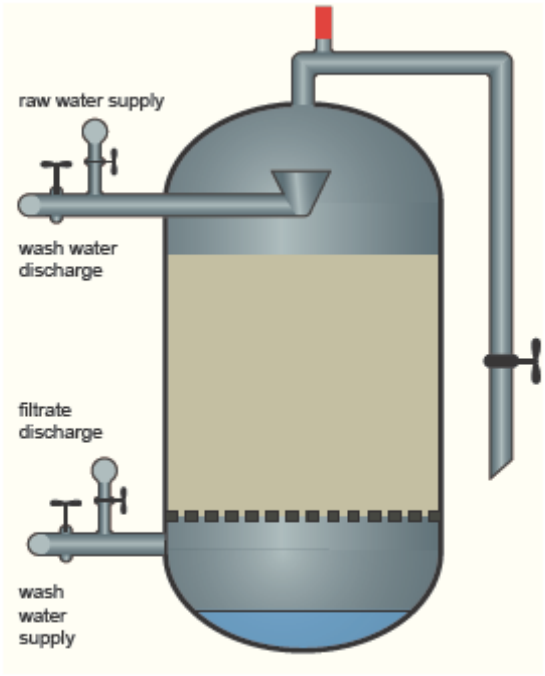



*Figure 24 - Principle of pressure filtration*

مرشحات الضغط تعمل بنفس الأسس التي تعمل بها المرشحات الرملية الشريعة غير انه في هذه الحالة يكون الوسط الترشيحي وقاع المرشح والمياه الخام الموجودة اعلي المرشح محاطة بغلاف لا ينفذ الماء و هذا ما يعني نظام مغلق يتم إجبار الماء على المرور خلال الوسط الترشيحي وهو ما يعني انه لا خوف من حدوث ضغط سالب وليس هناك أهمية للفقد في الضغط .

و هذا النوع من المرشحات يسمح باستخدام معدلات الترشيح السريعة فتكون من ٧ الي ٢٠ م/س .و بالتالي يمكن أن تكون مساحة المرشح صغيرة جدا وكذلك فترة تلامس المياه مع الوسط الترشيحي .

## الترشيح من أسفل الي اعلي

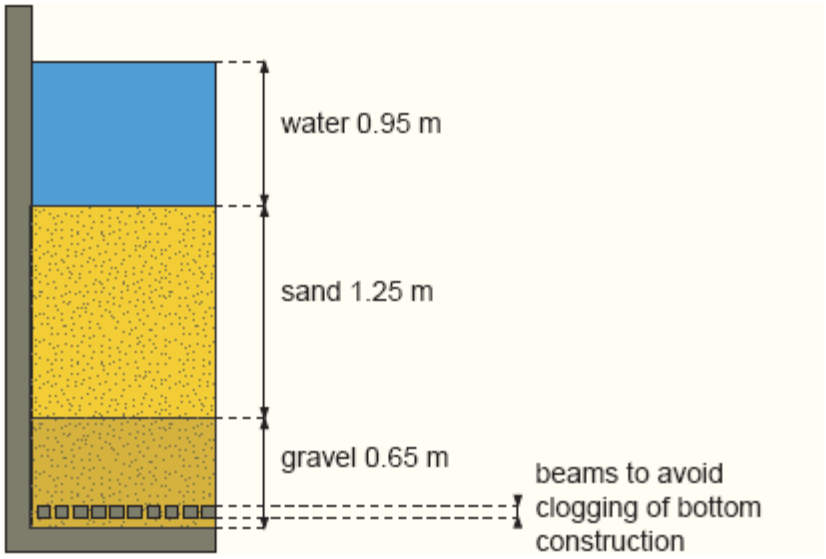



*Figure 28 - Principle of uplow filtration*

هو الترشيح ذو فيره التشغيل الأطول والجودة الأفضل بين المرشحات عندما يمر الماء من الوسط الخشن الي الوسط الترشيحي الناعم . في هذا النوع من المرشحات يكون الوسط الترشيحي الخشن أسفل الوسط الترشيحي الناعم و يسري الماء من أسفل الي اعلي , تتراكم معظم المواد العالقة بالطبقة السفلى و يكون معدل زيادة مقاومة الوسط بطيئاً وبزيادة المقاومة يزداد الضغط تدريجيا ويقل تدريجيا الى اعلى .

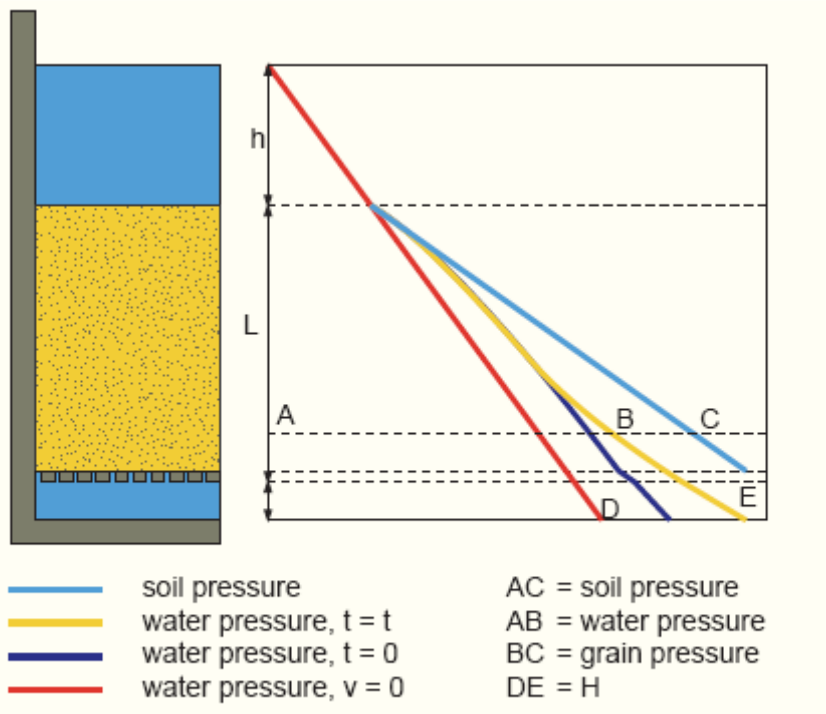



*Figure 29 - Pressure distribution in an upflow filter*

و للترشيح من اسفل الي اعلي مزايا كثيرة منها عدم حدوث الضغط السالب و الجودة العالية للمياه المنتجة وزمن الترشيح الطويل . و لكن له عيوب منها :

ان المياه العكرة تدخل من نفس مكان دخول مياه الغسيل مما قد يتسبب في انسداد الفواني في قاع المرشح وبالتالي عدم انتظام غسيل الوسط الترشيحي وبالتالي تسوء جودة المياه المرشحة .

قد يفقد بعض من الوسط الترشيحي أثناء عملية الترشيح عندما يزداد الضغط نتيجة انسداد الوسط الترشيحي مما قد يسبب تعويم الوسط الترشيحي و هذا الفقد قد يؤدي الي نقص ارتفاع الوسط الترشيحي وبالتالي تسوء جودة المياه المرشحة .

## الترشيح بواسطة الحجر الجيري ( limestone filtration )

وهذا النوع من الترشيح يستخدم في حالة المياه العدوانية ( aggressive water ) عندما يزداد نسبة حمض الكربونيك يتم امرار الماء خلال وسط ترسيحي من الحجر الجيري (calcium carbonate ) وعندها سيقل تركيز حمض الكربونيك و يزداد الاس الهيدروجيني

و يعمل الماء علي اذابة الحجر الجيري نتيجة هذا التفاعل .

$$CaCO_3 + CO_2 + H_2O \longleftrightarrow Ca^{2+} + 2 \cdot HCO_3^-$$

و لأن الوسط الترشيحي يذوب نتيجة هذه التفاعلات يجب ان يتم تعويض الفاقد منه بطريقة دورية .ويتم تعويض الفاقد عندما تصل نسبة الفاقد الي ١٠ % من الوسط الترشيحي , عند علاج المياه الجوفية العدوانية بواسطة هذا النوع من المرشحات بعد مرحلة المعالجة بالتهوية يمكن ان يتم إزالة الحديد و المنجنيز داخل المرشح .

## الترشيح المتواصل

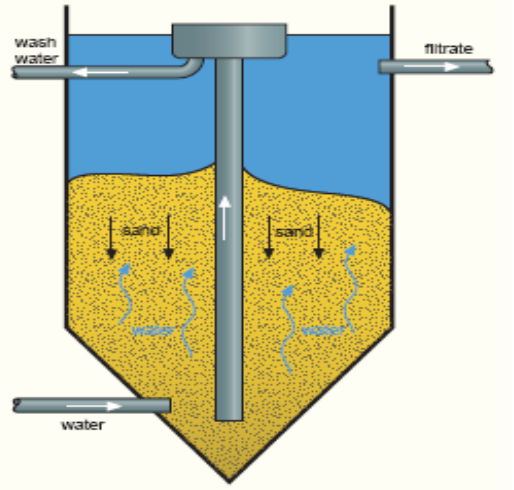



*Figure 30 - Schematic representation of a continuous filter*

في هذا النوع من المرشحات يتم إعادة تدوير الرمال و غسلها داخل المرشح وهو ما يعني ان عملية الغسيل مستمرة يدخل الماء الخام للمرشح من اسفل ويخرج الماء المعالج من اعلي من خلال هدار , الرمال المختلطة بالشوائب تنتقل الى اعلى وتحل محلها رمال نظيفة من المنطقة العليا و تتم عذه الحركة بواسطة طلمبة أو بواسطة ضغط الهواء , بناءا على معدل غسيل الوسط تكون جودة المياه المعالجة و ايضا كمية المياه المفقودة في الغسيل .معدل الترشبح في هذا النوع من المرشحات يتراوح من ١٤ الي ١٧ م/س .

مزايا هذا النوع من المرشحات مقارنه بالمرشحات الرملية السريعة

١. عملية المعالجة مستمرة
٢. عملية الغسيل مستمرة
٣. كمية الروبة المتجمعة اقل

عيوب هذا النوع من المرشحات مقارنة بالمرشحات الرملية الشريعة

١. كمية مياه الغسيل كبيرة
٢. تختلط مياه الغسيل مباشرة بالمياه المعالجة مما يزيد من مخاطر التلوث .

# المرشحات الجافة ( dry filtration )

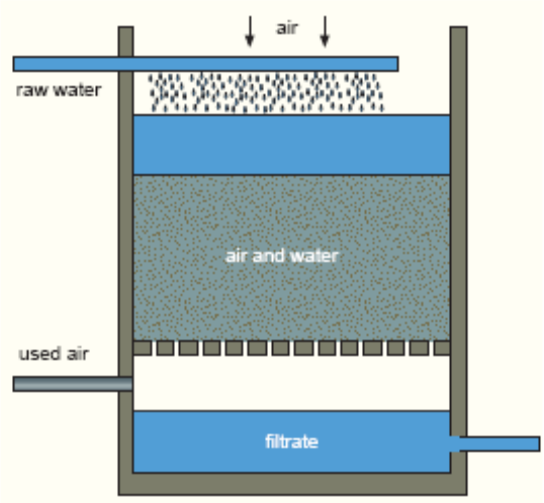



*Figure 31 - Schematic representation of a co-current filter*

يستخدم هذا النوع من المرشحات لمعالجة المياه التي تحتوي علي نسبة آمونيا عالية , ويتم تطبيق هذا النوع من المعالجة لمعالجة المياه الجوفية , ليتم اكسدة الأمونيا الي نترات يتطلب كمية كبيرة من الاكسجين ( 3.55 mg/l O2 per mg/l NH4+. )

في المرشح الجاف لا يوجد مياه خام اعلي الوسط الترشيحي و يمر الماء خلال الوسط الترشيحي الي قاع المرشح بينما يمر الهواء في نفس اتجاه المياه او في عكس الاتجاه , عملية انتقال الغازات تستمر بين الياه والهواء والوسط الترشيحي يمتلأ نسبيا بالمياه , سرعة الترشيح اعلي من المرشحات الرملية السريعة و سريان المياه بين حبيبات الوسط الترشيحي سريان دوامي مما يزيد من احتمال خروج الشوائب الملتصقة بالحبيبات مع المياه المعالجة .

لزيادة تعريض المياه للهواء يتم رش المياه من خلال فواني اعلي الوسط الترشيحي وهو ما يعمل علي تساوي توزيع المياه علي المرشح .

المرشح الجاف يزيل الأمونيا و ايضا الحديد و المنجنيز حيث تحدث عملية ازالة الحديد في الطبقة العليا من المرشح ( عمق المرشح من ٠.٥ الي ١.٥ م ) و تحدث بعد فترة عملية إزالة المنجنيز و الأمونيا , المرشح الجاف يمكن ان يتبع بمرشح رملي سريع لحجز الشوائب الناتجة من المعالجة بواسطة المرشح الجاف .

# المرشحات الرملية البطيئة

يستخدم هذا النوع من المرشحات عندما يكون الغرض من المرشح ازالة البكتريا والفيروسات كبديل لاستخدام وسائل التعقيم

المرشح الرملي البطيء يتميز بوسط ترشيحي ذو حجم فعال صغير ( ٠.٢ الي ٠.٦ ) و معدل الترشيح اقل من ١ م/س وبالتالي تحتاج نفس كمية المياه الي مساحات اكبر من الوسط الترشيحي مقارنة بالمرشحات الرملية السريعة .كما يتضح بالصورة

*Figure 33 - Difference in surface area between rapid filtration and slow sand filtration*

يحدث الترشيح في الطبقة العليا حيث تتكون الطبقة البيولوجية التي تعمل علي المعالجة .لتنظيف المرشح لا يتم غسله بل يتم كشط الطبقة العليا ( ١ سم ) .

لان المرشحات الرملية البطيئة تقع بعد المرشحات الرملية السريعة فان المرشح البطيء لا يتمك تحميله بالشوائب ولذا قد تمتد فترة تشغيله إلى سنوات .

## أنواع المرشحات الرملية السريعة

يمكن تصنيف المرشحات تبعا ل :-

١. عمق المنشأ الخرساني للمرشح حيث تنقسم المرشحات الي مرشحات عميقة ( deep filters ) و مرشحات ضحلة ( shallow filters ) وهو ما يدل علي كيفية التحكم في المرشح و دخوله أو إخراجه من الخدمة .
٢. يمكن تصنيف المرشح تبع لارتفاع المياه الخام فوق الوسط الترشيحي أثناء الترشيح فنقول ان المرشح ذو منسوب ثابت أو منسوب متغير .
٣. كما ينم تصنيف المرشح من حيث ثبات أو تغير معدل الترشيح فنقول مرشح ذو معدل ترشيح متناقص .

و هذه الفروق كلها تتصل بكيفية تشغيل المرشح أو التحكم في المرشح و لأنظمة التحكم في المرشحات أنواع عديدة وهي:

# أنظمة التحكم في المرشحات

أثناء تشغيل المرشحات الرملية السريعة التي تعمل بالجاذبية تتجمع الشوائب وتترسب داخل مسام الوسط الترشيحى مسببة زيادة فى مقاومة الوسط الترشيحى لحركة المياه من أعلى إلى أسفل ومع ثبوت باقي العوامل فإنه سوف يحدث انخفاض فى معدل الترشيح ,وقد يحدث انخفاض مماثل في معدل الترشيح عندما ينخفض منسوب المياه الخام أعلى الوسط الترشيحى.

وبالعكس عندما يرتفع منسوب المياه الخام أعلى الوسط الترشيحى أو ينخفض منسوب المياه المرشحة الخارجة من الوسط الترشيحى يزداد معدل الترشيح ٠

ونظراً لأهمية جودة المياه المرشحة فيجب الحفاظ على معدل الترشيح ثابتاً قدر الإمكان ويجب تفادى حدوث تغيرات فجائية فى معدل الترشيح , حيث قد تسبب الزيادة الفجائية فى معدل الترشيح خروج الشوائب الموجودة بالمياه الخام مع المياه المرشحة مؤثرة على جودة المياه المنتجة, وأيضاً مع حدوث انخفاض فجائى فى معدل الترشيح قد يسبب تكون فقاعات هوائية داخل الوسط الترشيحي و التى تحدث فجوات بالوسط الترشيحى تمر من خلالها الشوائب فتؤثر أيضاً فى جودة المياه الخارجة ٠

ومن هنا ظهرت الحاجة لوجود نظام تحكم فى معدل الترشيح وطبقاً للنظرية الرياضية للترشيح فإن المقاومة الهيدروليكية للوسط الترشيحى تتناسب عكسياً مع معدل الترشيح ٠

$$V = C\ (H1 - H2)$$

حيث H1 هو منسوب المياه الخام أعلى الوسط الترشيحى ٠

، H2 هو منسوب المياه المرشحة عند خروجها من المرشح ٠

، C هو ثابت يعتمد على حجم حبيبات الوسط الترشيحى ومواصفاتها وتوزيعها ومسامية وسمك الوسط الترشيحى ( فى حالة الوسط الترشيحى النظيف ) ٠

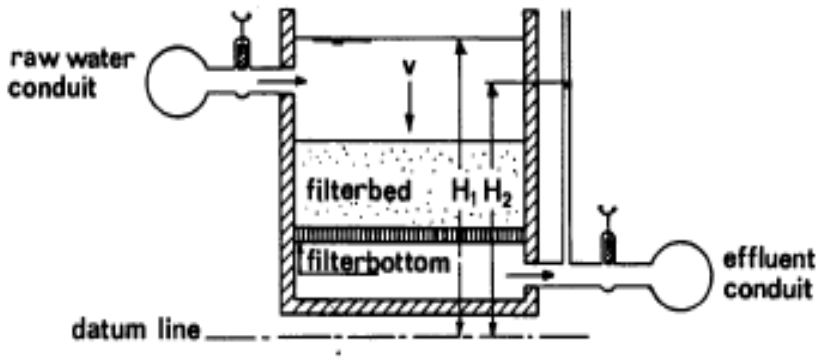


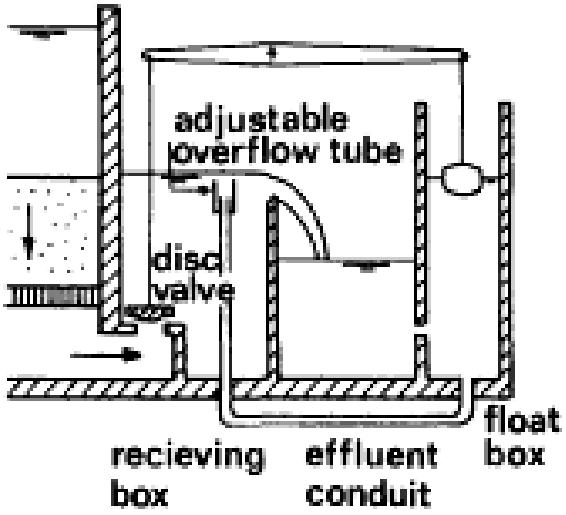

Fig. 4.8 Basic factors in filter control

ومع استمرار عملية الترشيح تقل مسامية الوسط الترشيحى تبعاً لكمية الشوائب المترسبة مسببة انخفاض فى قيمة الثابت ( C ) وعندها يتطلب الأمر زيادة الفارق بين H1، H2 للحفاظ على معدل الترشيح ( V ) ثابتاً ٠

وللتحكم فى معدل الترشيح يجب إدراج فاقد إضافي للضغط فى خط دخول المياه العكرة أو خروج المياه المرشحة وبضبط الفقد فى الضغط يمكن الحفاظ على تصرف المياه العكرة أو خروج المياه المرشحة ثابتاً عند قيمة معينة ٠

وفى الأساس يمكن إجراء هذا الضبط يدوياً ولكن مع قصر مدة تشغيل المرشح فإنه تحدث تغييرات سريعة تتطلب مراقبة مستمرة وعمل تحكم أوتوماتيكي بواسطة وسائل ميكانيكية أو هيدروليكية أو ٠٠٠٠٠

قديماً كان كل مرشح مزوداً بوسائله الخاصة بالتحكم والتي تطورت بمرور الوقت كما بالشكل : –



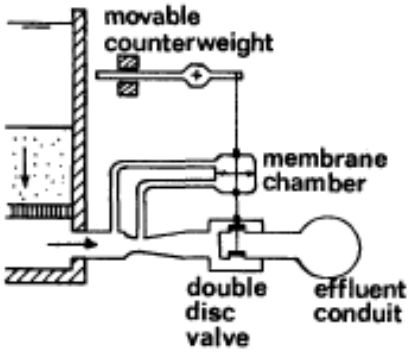




Fig. 4.10 Venturi rate controller with mechanical operation

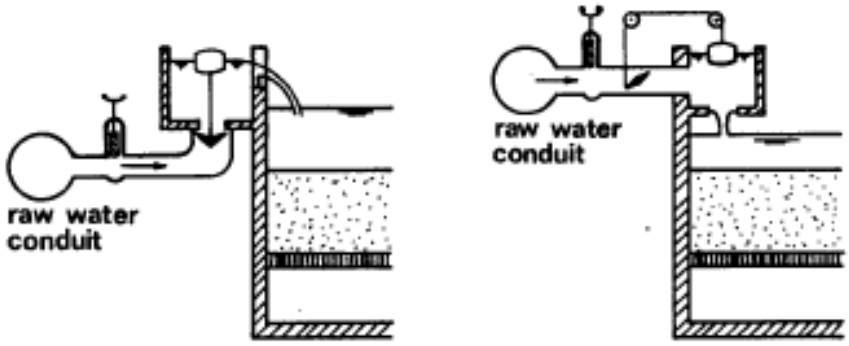



Fig. 4.11 Open filter controllers in raw water influent lines

عندما تزداد الحاجة لضرورة تطبيق التحكم فى تصرف المياه الخام فإن المنشأ المغلق فى شكل ( ٤-١٠ ) يمكن اللجوء إليه بدون أي تغيير بينما يبين شكل ( ٤-١١ ) إثنان من احتمالات تطبيق المنشأ المفتوح ، حيث تسرى المياه الخام من قناة الدخول إلى المرشح ومن ثم تسرى من على جدار ثابت أو من خلال فتحة معايرة إلى المرشح ٠

فى هذه الحالة فإن العامل المؤثر فى التوزيع هو ضغط المياه الخام ٠٠٠ هذا التغير فى ضغط المياه الخام صغير جداً يمكن إهماله ومنسوب المياه داخل المرشح يظل ثابتاً ويمكن التحكم فيه من خلال محبس دخول المياه الخام والمتصل بعوامة على منسوب المياه الخام وهذا النظام للتحكم المنفرد لكل فلتر له مساوئه حيث يجب ضبط كل مرشح بمفرده وأيضاً لارتفاع ثمنه وأيضاً لأنه يسبب فقد كبير فى الضغط حتى فى حالة كونه مفتوحاً تماماً ٠

هذه هى الأسباب التي تجعل استخدام أنظمة التحكم المنفردة نادرة الاستخدام فى منشآت المياه الحديثة ٠

وفى شكل ( ١٤ – ١٢ ) تقوم قناة دخول المياه الخام بخدمة بطارية من المرشحات مع الأخذ فى الاعتبار أنه يجب أن تكون قناة الدخول ذات مساحة مقطع كبيرة و التي تقلل من الفقد نتيجة الاحتكاك أو التيارات الدوامية وكذلك تأثير إرتفاع منسوب المياه الخام يمكن إهماله ٠

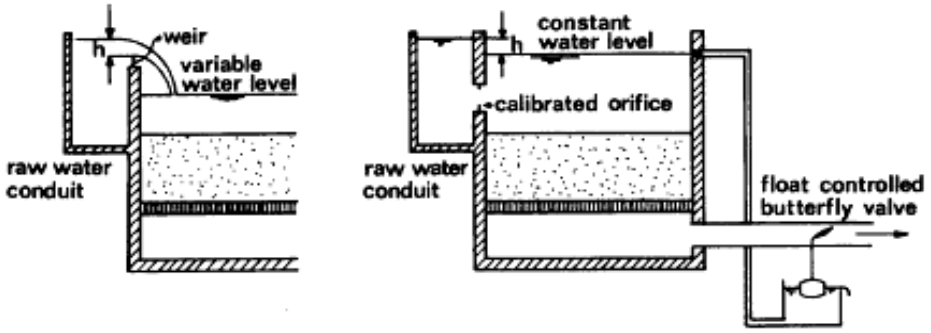



Fig. 4.12 Equal supply of raw water to all filtering units

وأمام كل مدخل للمياه الخام هدار وعندما يتم ضبط الهدارات بنفس المنسوب لكل المرشحات فإن كل المرشحات تستقبل نفس كمية المياه وهو ما يتطلب جهاز تحكم إضافي ولكنه يعطى ميزة عدم تأثير تغير مقاومة الفلتر ( h ) على معدل الترشيح ٠ وعامة فإن معدل الترشيح يزداد بزيادة تصرف المياه الخام الداخلة للمرشح و مع ارتفاع منسوب المياه الخام بقناة الدخول وزيادة قيمة ( h ) ٠

فى حالة السحب المتساوى للمياه المرشحة من كل المرشحات ( شكل ٤-١٣ ) فإن قناة الخروج تكون ذات مساحة مقطع كبيرة لضمان منسوب منتظم للمياه وعندما يكون منسوب المياه فى غرفة العوامة عند مستوى ثابت لجميع المرشحات فإن جميع فتحات الخروج سوف تمرر نفس كمية المياه وعندما يزداد استهلاك المياه المرشحة وينخفض منسوب المياه فى قناة الخروج تزداد قيمة المقاومة ( h ) ٠

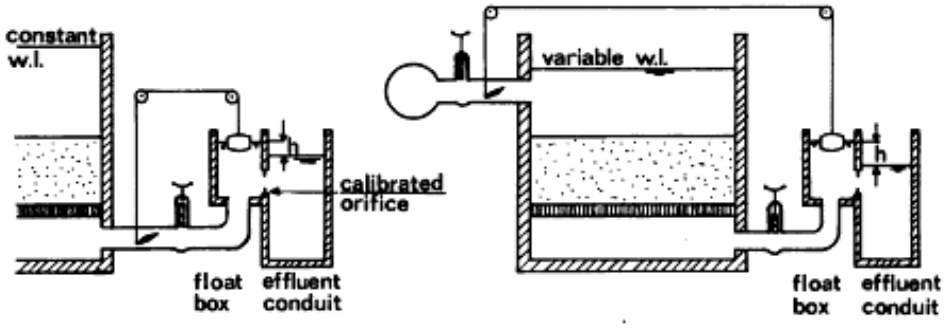



Fig. 4.13 Equal abstraction of filtered water from all filtering units.

وفى حالة خروج أحد المرشحات للغسيل يتم توزيع كمية المياه الداخلة لذلك المرشح على باقى المرشحات بدون الحاجة إلى أجهزة تحكم إضافية .

لا يمكن تحقيق معدل ترشيح متساوي لكل المرشحات سواء باستخدام أجهزة تحكم فى التصرف أو بتقسيم كمية التصرف بين المرشحات.

ومن أجل التحكم الكامل فى المرشح يجب أن يصاحب التحكم فى التصرف أيضاً فى منسوب المياه سواء كان منسوب المياه الخام فوق الوسط الترشيحى أو منسوب المياه المرشحة أسفل الوسط الترشيحى .

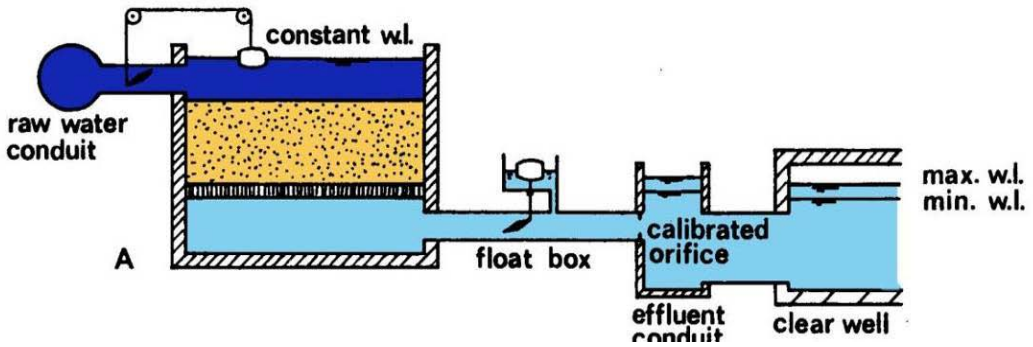
إن التحكم فى المرشح يساعد على جعل تشغيل المرشح مستقل عن التغيرات فى كميات المياه الخام الواردة أو كميات المياه المرشحة المستهلكة .ومن ناحية آخرى فإن التحكم في المرشح يجعل زيادة مقاومة الوسط الترشيحى– المصاحب لانسداد المرشح أثناء فترة تشغيل المرشح – أكثر استقرارا .

ويتم التحكم فى المنسوب بواسطة إدراج فقد إضافي فى الضغط بخط الدخول أو الخروج ليعمل على ضبط هذا الفقد فى الضغط محافظاً على فرق المنسوب بين دخول المياه وخروجها .

وتعمل أنظمة التحكم فى التصرف والمنسوب سوياً على نفس الخط وهناك أربع بدائل للتحكم فى المرشحات : –

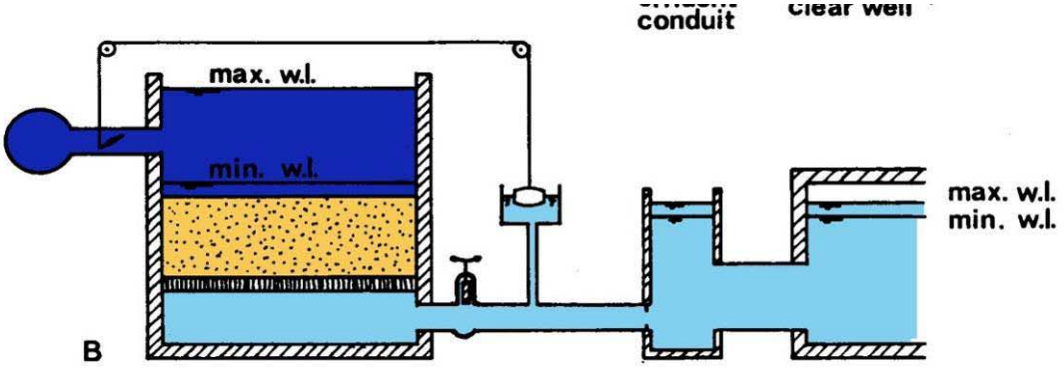
١. التحكم فى كمية تصرف المياه الخارجة مع التحكم فى منسوب المياه الخام .
٢. التحكم فى كمية تصرف المياه الخارجة مع التحكم فى كمية تصرف المياه الداخلة بإشارة من منسوب المياه المرشحة .
٣. التحكم فى كمية تصرف المياه الخام مع التحكم فى كمية تصرف المياه الخارجة بإشارة من منسوب المياه الخام .
٤. التحكم فى كمية تصرف المياه الخام مع التحكم فى كمية تصرف المياه المرشحة بإشارة من منسوب المياه المرشحة .

**وتمثل الأشكال الآتية هذه البدائـل .**



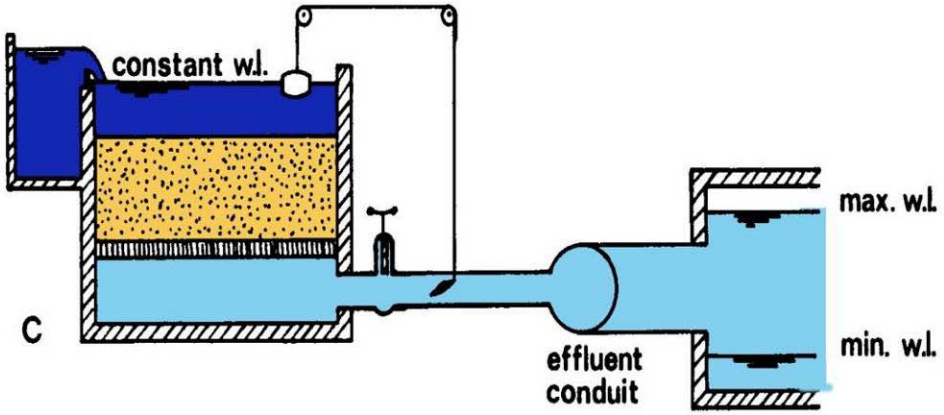

**في النموذج (A)**

يتم الحفاظ على ثبات منسوب المياه الخام بواسطة عوامة تتحكم في محبس فراشة موجود على خط دخول المياه العكرة ونظام مماثل آخر يقوم بالحفاظ على منسوب المياه المعالجة على ماسورة الخروج ليظل المنسوب ثابتا لكل المرشحات وعندما تكون قناة الخروج ذات مساحة مقطع كبيرة بالتالي يصبح منسوب المياه منتظما على طول الماسورة ،و الفتحة المعايرة في نهاية ماسورة الخروج سوف تقوم بتصريف كميات متساوية من المياه حتى عندما يكون منسوب المياه بماسورة الخروج منخفضا .



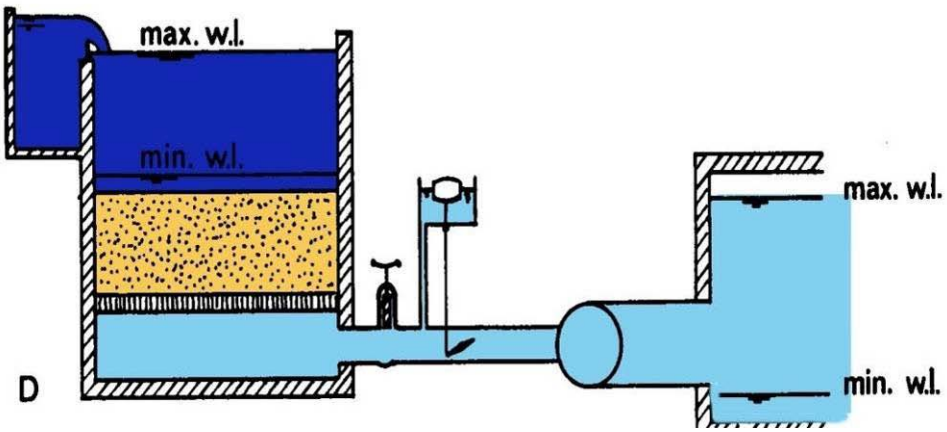

**في النموذج (B)**

يتم التحكم في منسوب المياه المرشحة بواسطة عوامة تتحكم في محبس على ماسورة دخول المياه العكرة ولأن منسوب المياه منتظم في قناة خروج المياه المرشحة فان فتحات المعايرة في نهاية مواسير الخروج لتقوم أيضا بتصريف كميات متساوية من المياه حتى إذا انخفض منسوب المياه في الخزان أو في قنوات خروج المياه المرشحة




**في النموذج (C)**

يتم الحفاظ على تصرف المياه الخام ثابتا بواسطة عوامة تتحكم في محبس فراشة موجود على ماسورة خروج المياه المرشحة ، وعندما تكون قناة دخول المياه العكرة ذات مساحة مقطع كبيرة فان منسوب المياه يكون منتظما على طول قناة الدخول حيث يتم ضبط الهدارات على جميع أماكن دخول المياه العكرة للمرشحات وتكون على منسوب واحد لتضمن تساوي تصرف المياه لكل الفلاتر حتى إذا ارتفع منسوب المياه في ماسورة دخول المياه العكرة .




**في النموذج (D)**

يتم الحفاظ على منسوب المياه المرشحة بواسطة عوامة تتحكم في محبس فراشة على خط خروج المياه المرشحة بينما تقوم الهدارات – التي تم ضبطها على منسوب واحد – بقناة الدخول لضمان تساوى كمية تصرف المياه العكرة لكل مرشح حيث تمتاز قناة الدخول بمساحة مقطع كبيرة ومنسوب مياه منتظم .

**وبالإشارة إلى الأنظمة المختلفة المذكورة يجب ملاحظة الاتي:-**

أنه بالنسبة للبديل (B) ، (D) فإن منسوب المياه المرشحة يظل ثابتاً على مسافة صغيرة فوق منسوب الوسط الترشيحى وأثناء عملية الترشيح تزداد مقاومة الوسط الترشيحى ويتم التغلب على ذلك بارتفاع منسوب المياه الخام ٠٠ وعند ذلك يستحيل حدوث ضغط سالب ولكن يجب أن يكون المنشأ الخرساني للمرشح كبيراً بدرجة كافية ( مما ترتفع معه تكلفة المنشأة ) .

وفى البديل (A) ، (C) يمكن منع حدوث ضغط سالب عندما يكون منسوب المياه الخام مرتفعاً بمسافة كبيرة أعلى الوسط الترشيحى و تنتهي فترة تشغيل المرشح عندما ينخفض منسوب المياه المرشحة .

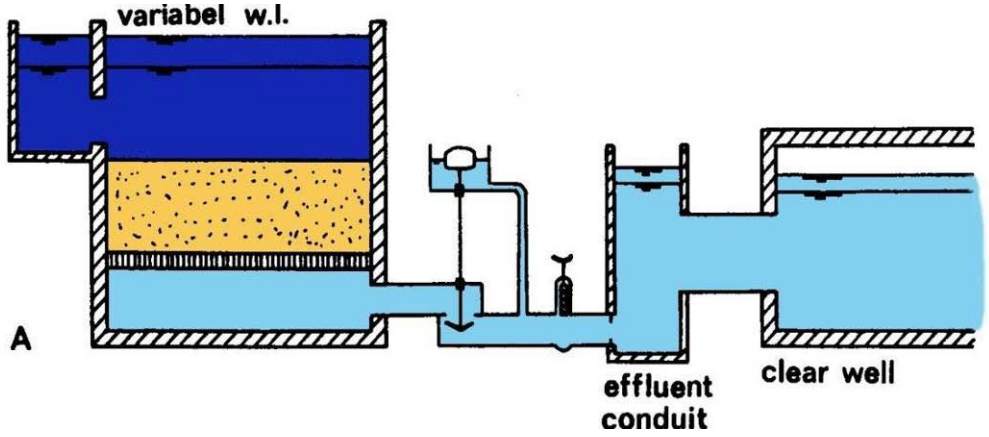
وبهذه المنشآت فقط ( المرشحات ذات المنسوب الثابت ) يمكن الحصول على منسوب ضحل للمياه الخام ( ٢٥ سم : ٤٠ سم ) مثلاً وعندها سوف يتكون ضغط سالب كبير ولكن من ناحية اخرى فإن ارتفاع المنشأ الخرسانى للمرشح أقل و بالتالى تكلفة الإنشاء أقل مما يجعل هذا التصميم مفضلا فى الشركات المتخصصة فى بناء المرشحات .

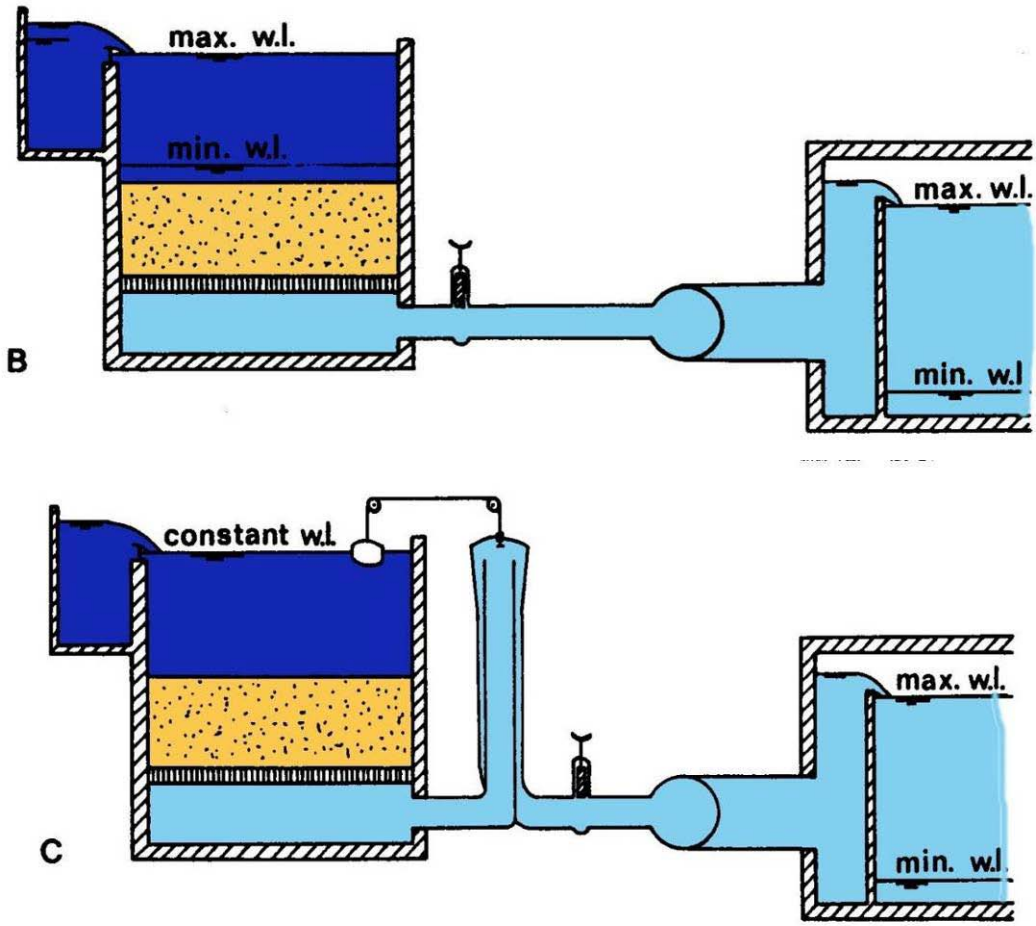
وفى البديل (C) ، (D) يتم التحكم فى كمية تصرف المياه الخام بينما منسوب المياه المرشحة فى الخزان قابل للزيادة بنسبة كبيرة مسبباً قدرة تخزينية كبيرة ومن ناحية أخرى فى البديل (A) ، (B) يتم التحكم فى كمية تصرف المياه المرشحة بينما منسوب المياه المرشحة فى الخزان قد يتغير بنسبة صغيرة متطلباً متابعة إضافية لموازنة تصرف المياه الخام وإستهلاك المياه المرشحة لكل فترة زمنية صغيرة ( من ٠.٥ : ١ ساعة ) .

## أي البدائل السابقة يمكن اختيارها وتطبيقهـا ؟

هذا يعتمد بالدرجة الأولى على الظروف المحيطة وأيضاً على ما يفضله المعمم .

وكقاعدة عامة يجب ذكرها فإنه يجب الاهتمام بمنع تجمع الندف فى الوسط الترشيحى والتى قد تجعل تشغيل المرشح مهمة صعبة وهو ما يضع الحدود الخاصة لاستخدام التحكم فى كمية المياه الخام طبقاً لزيادة مقاومة الوسط الترشيحى كما يضع حدوداً آخري لتطبيق نظام منسوب المياه الخام المتغير مما يجعل البديل الأول مقترح وحيد






ويمكننا الحصول أيضاً على نتائج أفضل بناءاً على التصميم المذكور بالشكل ( ٤ – ١٥ أ ) حيث يكون أرتفاع المياه الخام فوق الوسط الترشيحى كبيراً ويتم التحكم فى منسوبه بواسطة تغيير قدرات طلمبات المياه العكرة وللتغيير فى هذا المنسوب مسافة ½ متر يعطى كفاءة تخزينية عالية كافية لتشغيل طلمبات المياه المرشحة بصفة خاصة وأيضاً يتسبب فى أرتفاع منسوب المياه الخام فى منع حدوث ظاهرة الضغط السالب ويمكن الحصول على أبسط الحلول ( البدائل ) باستبدال التحكم فى منسوب المياه المرشحة بشكل ( ١٤ – د) بهدار ثابت بخط الخروج وبدون أى أجزاء متحركة وبدون احتمال لحدوث أعطال مما يجعل هذا التعميم أكثر انتشارا للمرشحات الرملية السريعة قبل البطيئة .

إن احتمال ارتفاع منسوب المياه الخام احتمال غمر الحوائط الخاصة بالمرشح وأبتلالها مما مما يجعلها معرضة لترسيب الطمي والحديد و المنجنيز وغيرها من المواد على الحوائط مما يجعلها فى حاجة إلى النظافة المستمرة حتى لا تؤثر على العوامل الصحية الخاصة لمحطات مياه الشرب .

وبالإشارة إلى ما سبق فإن البديل الثالث ( 14 - 4 ) – ( 14 c - 4 ) أكثر ملائمة .

ملحوظـة : -

فى شكل ( 14 c - 4 ) تم استبدال محبس الفراشة بسايفون به فتحة دخول هواء فى أعلاه مما يقلل تصرف المرشح وهذا السايفون يتميز بأنه رخيص الثمن وعمره الإفتراضى طويل ولكن يجعل تشغيله منفصل عن تغيير منسوب المياه المرشحة بالخزان يجب عمل هدار إضافى ، هذا الهدار يعمل على منع الضغط السالب بالوسط الترشيحى ويقوم بعمل تهوية دخول جيدة للخزان .

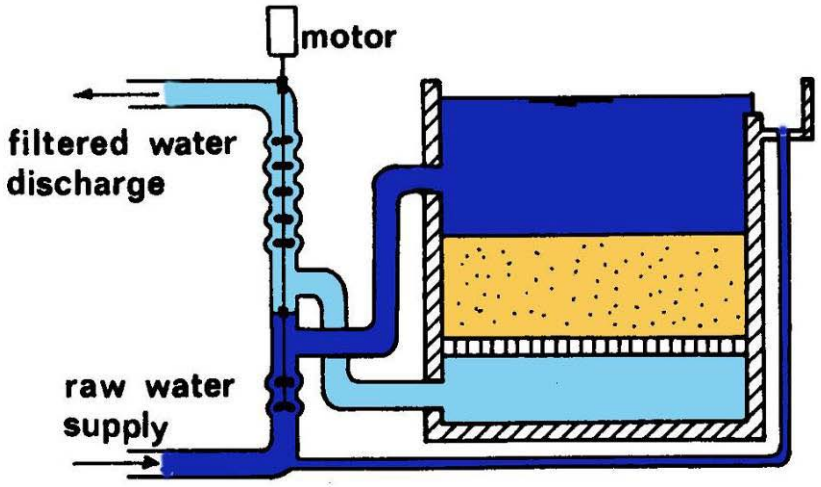
يجب تثبيت منسوب المياه الخام عندما يحدث تغير فى أرتفاع المياه أعلى الوسط الترشيحى مما يتطلب فترة زمنية كافية قبل الدخول على المرشح .

فى حالة معدلات الترشيح العالية تكون هذه الفترة الزمنية قصيرة مما يتطلب إرتفاع منسوب المياه الخام عن الوسط الترشيحى بمسافة كبيرة .

إن البديل ( 14 b - 4 ) نادراً ما يستخدم ولكنه يتميز بوجود بعض المميزات عندما تدخل المياه الخام للمرشح تحت ضغط كبيرومتغير وغنياً عن البيان وأن شكل ( 15-4 )، (14-4) تعرض بعض النماذج التى تبين طرق التحكم فى المرشحات .

وبالنسبة للتفاصيل فيوجد عدد كبير ومتنوع من أساليب التحكم فى المرشحات ومتاح تجارياً وإلى يومنا هذا لاتزال بعض الحلول الجديدة التى تظهر وتتطور بصورة مستمرة .

معظم هذه البدائل قد لاتكون سهلة وبسيطة فى الانشاء كما فى شكل ( ٤-١٤ ) ولكن قد يستثنى من ذلك نظام التحكم فى الطلمبات فى شكل ( ٤-١٦ )



حيث يتم تشغيل طلمبات بالمياه العكرة والمياه المرشحة من نفس المحرك الكهربى بينما تحت كل ظروف تشغيل فإن قدرة طلمبات المياه العكرة أكبر من قدرة طلمبات المياه المرشحة وأى زيادة فى كمية المياه الخام يتم صرفها خلال الفايضة لتعود إلى دخول المياه العكرة وهنا يعنى فقدان فى الطاقة ولكن تظل الزيادة فى تكاليف التشغيل يمكن إهمالها وفى شكل ( 4-15a ) يعتمد معدل الترشيح على استهلاك المياه المرشحة ويزداد عندما ينخفض منسوب المياه بالخزان وفى قناة الخروج ما يسبب فقدان فى الطاقة فى نهاية فتحة خروج المياه المرشحة من المرشحات ولايوجد أدنى تأثير لتصرف المياه العكرة على معدل الترشيح ولكن فقط تغير منسوب المياه العكرة فوق الوسط الترشيحى .

وفى حالة الرغبة فى العمل بنفس البديل ولكن بمعدل ترشيح يغتمد على تصرف المياه الخام ويجب إضافة جهاز تحكم جديد .

وفى شكل ( 4-17a ) فإن الزيادة فى تصرف المياه الخام يرفع منسوب المياه الخام فى بطارية دخول الفلاتر وبفتح محبس الفراشة فى ماسورة الخروج بين قناة خروج المياه المرشحة بالبطارية وبين الخزان مسبباً انخفاض فى منسوب المياه فى قناة خروج المياه المرشحة مما يزيد من معدل الترشيح .

وبنفس الطريقة السابقة وحتى يتم تحقيق اتزان القدرة وكمية تصرف المياه الخام فإن الحل العكسى يظهر فى شكل ( 4-15c ) عندما يتغير استهلاك المياه المرشحة يتغير منسوب المياه بالخزان ولكن بدون التأثير على معدل الترشيح والذى يعتمد فقط على كمية المياه الخام الواردة ٠٠ وبواسطة جهاز تحكم إضافى كما فى شكل ( 4-17b ) فإن الزيادة فى استهلاك المياه المرشحة تسبب انخفاض فى منسوب المياه بالخزان ويفتح محبس الفراشة على ماسورة دخول المياه الخام إلى قناة دخول المياه الخام لبطارية الفلاتر ، منسوب المياه بقناة دخول المياه الخام عندها سيرتفع منسوب المياه الخام وسيبدأ هدار دخول المياه الخام فى تمرير مزيد من المياه ليرتفع منسوب المياه فى الخزان حتى يتزن مع أستهلاك المياه المرشحة .

وفى حالة ثبات تصرف المياه الخام أو ثبات أستهلاك المياه المرشحة فإن أنظمة التحكم فى هذا الجزء فقط تعمل على ثبات معدل الترشيح ولسوء الحظ فلإن هذا لايمكن يتحقق بصورة كاملة ليظهر فى صورة مما يؤثر فى جودة المياه المنتجة بينما نظام ( 4-15A ) يطبق ومن المعروف بديهياً أن وزن العوامة وقرص المحبس وذراع التوصيل وأيضاً إجمالى القوى الهيدروليكية المؤثرة على قرص المحبس تعمل كلها لتعاكس قوة

الطفو وفى الحقيقة قد يحدث التضاد وهذا الأختلاف قد يحدث بسبب الاختلاف بين ذراع التوصيل والجلبة بصندوق العوامة أو المحبس وعندها قد يزداد استهلاك المياه المرشحة على تصرف المياه العكرة مسبباً أنخفاض منسوب المياه العكرة تدريجياً وتتسبب قوى الاحتكاك المذكورة سابقاً لبقاء قرص المحبس فى وضعه الأصلى وعندها ينخفض منسوب العوامة وينخفض أيضاً معدل الترشيح .

وفى هذه الحالة تقوم قوة طفو العوامة تدريجياً بتقليل الفارق حتى تزداد قيمتها على قيمة قوة الاحتكاك المقاومة وعندها تحرك العوامة فجأة لأسفل .

وحركة قرص المحبس لأسفل تعمل على ضبط فتحة المحبس بزيادة مفاجئة لمعدل الترشيح بصورة لايمكن تفاديها وهذه الظاهرة بظاهرة ( hunting ) ولها أيضاً عدد من الأسباب تختلف بأختلاف قوى الاحتكاك وفى شكل ( 4-15A ) إذا لم يتم ضبط قرص المحبس بطريقة صحيحة تتولد دوامات والتى قد تسبب ترددات ( ذبذبات ) تزيد وتقلل من معدل الترشيح وكثيراً من أساليب التشغيل الغير مرضية لمحطات المياه بسبب أخطاء يمكن أن نرجعها إلى أجزاء صغيرة كالمذكورة سابقاً .

إن معدل الترشيح يجب أن يتغير مع كميات المياه الخام أو استهلاك المياه المرشحة بالزيادة أو النقصان وتشير الخبرات السابقة أن هذه الأخطاء فى التشغيل لاتؤثر بصورة مباشرة على جودة المياه المنتجة ما دامت نسبة الزيادة أو النقصان فى حدود ٣% فى الدقيقة وفى شكل ( A10-4 ) يتم ذلك أوتوماتيكيا عندما تزداد قدرة طلمبات المياه المرشحة بصورة فجائية فإنها تتسبب فقط فى انخفاض منسوب المياه فى الخزان تدريجياً .

## متي يخرج المرشح من الخدمة ؟

١. إذا ارتفع منسوب المياه الخام الي أقصى الحدود المسموح بها معلنا انسداد المرشح .
٢. إذا وصل الفقد في الضغط بين المياه أسفل الوسط الترشيحي والمياه اعلى الوسط الترشيحي الي الحد المسموح به قبل أن يصل الضغط اسل الوسط الترشيحي الي القيمة السالبة .
٣. إذا ساءت نتائج المياه المرشحة
٤. إذا قل تصرف المياه المرشحة عن الحد المسموح .

وعندما يصل المرشح الي احد هذه الحالات يتم إخراجه من الخدمة وغسله .

فما هي إجراءات الغسيل و كمية التصرف الواجبة لغسل المرشح ؟

## خطوات الغسيل

١. إيقاف دخول المياه العكرة
٢. يتم تخفيض منسوب المياه بالمرشح حتى يصل اعلي الوسط الترشيحي ب ٢٠ سم.
٣. يتم غلق محبس خروج المياه المرشحة ومحبس الصرف .
٤. يتم فتح محبس خروج صرف مياه الغسيل .ثم تبدأ إجراءات الغسيل

| المرحلة | المادة المستخدمة في الغسيل | الهدف | الزمن | السرعة |
|---|---|---|---|---|
| ١ | هواء أو ماء | بدء دخول المادة المستخدمة في الغسيل في الاتجاه المعاكس | من ١ الي ٢ دقيقة | ٣٠-٦٠ م / س |
| ٢ | هواء + ماء | عمل احتكاك لفصل الجزيئات العالقة ( scoring ) | من ٢ الي ٥ دقائق | ٣٠ . ٦٠ م/ساعة |
| ٣ | ماء يزداد تصرفة تدريجيا | طرد الهواء من الوسط الترشيحي | ١ +١ +١ دقيقة | ١٠.٢٠ .٣٠ م/س |
| ٤ | ماء فقط | طرد المواد العالقة | من ٥ الي ١٥ دقيقة | حتى يصل تمدد الوسط الترشيحي الي النسبة المطلوبة |
| ٥ | ماء يقل تصرفه تدريجيا | لتعود الجزيئات كل الي مكانة حسب الحجم والكثافة | إجمالي ٤ دقائق | ١٠-٢٠-٣٠ م/س |

مما سبق نري ان لكل مرحلة من مراحل الغسيل هدف يجب تحقيقه و اهم هذه الاهداف ان يتم تمدد الوسط الترشيحي لانه بدون تمدد الوسط الترشيحي تكون عملية الغسيل ما هي الا عملية ترشيح عكسي فقط

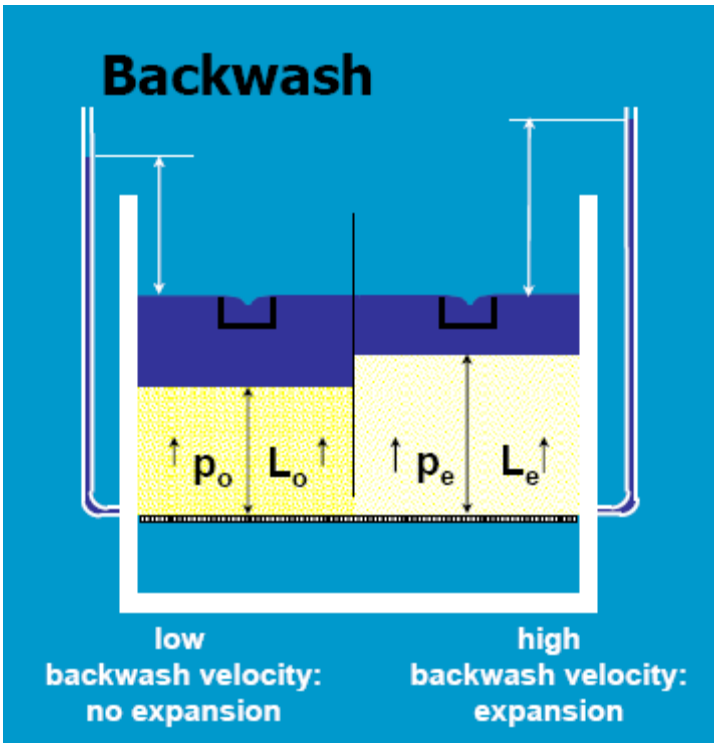
تعتمد نسبة تمدد الوسط الترشيحي اللازمة للغسيل الصحيح على حجم حبيبات الوسط

عندما تكون القطر الفعال ٠.٨ مم تكون نسبة التمدد = ١٥-٢٠%

عندما يكون القطر الفعال ١ مم تكون نسبة التمدد = ١٠%

حيث E هي نسبة تمدد الوسط الترشيحي .

$$E = \frac{L_e - L_o}{L_o}$$




لتحديد تصرف مياه الغسيل الصحيحة التي تحقق هذه النسبة يجب ان نطبق معادلة كارمون كازوني

$$V^{1.2} = \frac{g}{130 * v^{0.8}} * [\frac{\rho_f - \rho_w}{\rho_w}] * \frac{P_e^{3} * d^{1.8}}{(1-P_e)^{0.8}}$$

$$p_e = \frac{p_0 + E}{1 + E}$$

حيث

| | |
|---|---|
| filter bed height during expansion | $L_e$ |
| filter height during filtration | $L_0$ |
| porosity during filtration | $p_0$ |
| porosity during expansion | $p_e$ |
| bed expansion | $E$ |
| head loss | $H_{max}$ |
| water density( kg/m3) | $\rho_w$ |
| filter media density( kg/m3) | $\rho_f$ |
| kinematic viscosity | $\nu$ |
| velocity (m/hr) | $V$ |

و مما سبق نري ان تصرف مياه الغسيل التي تحقق نسبة التمدد المطلوبة لضمان تمدد الوسط الترشيحي تعتمد علي مواصفات الوسط الترشيحي من مسامية وكثافة وحجم فعال وكذلك علي ارتفاع الوسط الترشيحي و لزوجة المياه التي ترتبط بدرجة الحرارة .

و يتم غسيل المرشحات بواسطة طلمبات غسيل المرشحات أو بواسطة خزان علوي يوفر الطاقة المطلوبة للغسيل .

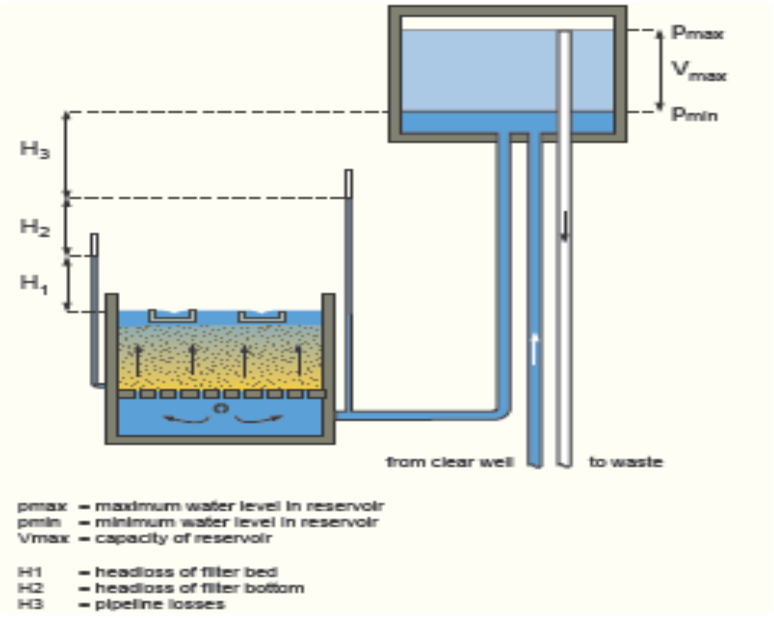



*Figure 13 - Elevated water reservoir as backwash system*

يجب ملاحظة انه عند غسيل المرشح ذم الوسط الترشيحي المزدوج يجب ملاحظة اختلاف نسبة التمدد لكل طبقة من الوسط الترشيحي و يجب ان تحقق كمية تصرف مياه الغسيل كلا النسبتين ولذلك يجب حساب كمية التصرف بدقة و كذلك عند تصميم المرشح اختيار حجم الحبيبات بحيث تحقق هذه الشروط .

## مشاكل المرشحات

علي الرغم من بساطة عملية الترشيح إلا أن كثير من المشكلات قد تحدث و منها

في حالة عدم انتظام الوسط الترشيحي يتم توزيع و تقسيم الحبيبات خلال عملية الغسيل فتكون الحبيبات الصغر و الأقل وزنا بالطبقة العليا للمرشح و تبقى الحبيبات الكبيرة ذات الوزن الأكبر بالطبقة السفلى للمرشح وهو مما يسبب سرعة انسداد المرشح نتيجة انسداد الطبقة العليا فقط .

في حالة عدم الغسيل الجيد وقلة تصرف مياه الغسيل لا يحدث تمدد للوسط الترشيحي وبالتالي لا يتم إزالة كل الشوائب من الوسط الترشيحي وبالتالي تقصر فترة تشغيل المرشح وبالتالي تزداد كميات مياه الغسيل المفقودة .

في حالة عدم انتظام توزيع مياه الغسيل نتيجة سوء تصميم أو انسداد الفواني بقاع المرشح لا يتم غسيل كل الوسط الترشيحي مما يسبب تكون الكتل الطينية التي تسبب أيضا حدوث شروخ بالوسط الترشيحي و سوء نتائج المياه المرشحة .

Figure 17 -Nozzle

Figure 20 - Result of a poorly designed filter bottom

بسبب عدم انتظام هدارات دخول المياه العكرة يمكن أن تتسبب في دخول معظم المياه من مكان معين وبذلك يعمل جزء من الوسط الترشيحي فقط وبالتالي تزداد معدلات الترشيح وتسوء نتائج المياه المرشحة .

وبسبب عدم انتظام هدارات خروج مياه الغسيل تخرج مياه الغسيل كلها من مكان واحد وبالتالي يتم غسيل جزء من الوسط الترشيحي فقط و تترسب الشوائب في باقي الوسط , وهو ما يؤدي الي تكون البكتريا والطحالب وسوء نتائج المياه المرشحة ثم حدوث شروخ بالوسط الترشيحي .

وبسبب سوء التشغيل قد يتعرض سطح الوسط الترشيحي الي الهواء لانخفاض منسوب المياه اعلي المرشح بعد الغسيل مم يسبب وجود فقاعات هواء داخل الوسط الترشيحي مما يسبب سرعة الانسداد و من ناحية أخري نقوم المياه المندفعة الي سطح والوسط الترشيحي بحفر سطح الوسط والخروج من قاع المرشح دون حدوث عملية الترشيح و هو مما يؤدي الي سوء نتائج المياه المرشحة .

وبسبب كسر في قاع المرشح قد يحدث فقد في الرمال مما يؤثر على جودة المياه المنتجة .

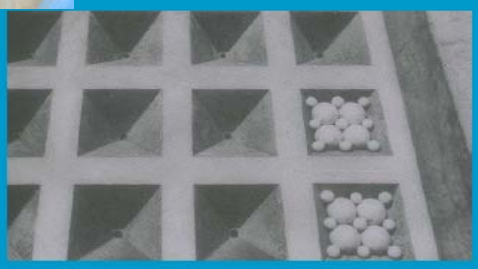
# قاع المرشح

لا شك ان مشكلات المرشح قد يشكل قاع المرشح عاملا مشتركا لها ولذلك وجب التعرف على انواع قاع المرشحات

مرشح ذو القاع المزدوج و الفواني.

المرشح ذو البلاطات الخرسانية. ( M block )

المرشح ذو شبكة المواسير المثقبة



# الملحق الأول

# Annex 1

# AERATION

## Description of Aeration

Aeration is the process of introducing air and water in contact. A common example of aeration occurring naturally is a stream tumbling over rocks. The resulting turbulence brings the air and water into close contact and the air dissolves into the water
this process can he used to remove several unwanted constituents from water

## How aeration removes or modifies constituents

In water treatment, the aeration process brings water and air into close contact or thin sheets of water to the air (water into air), or by introducing small bubbles of air and letting them rise through the water (air into water) . For both procedures. the processes by which aeration accomplishes the desired results are the same:

• Sweeping or scrubbing action caused by the turbulence of water and air mixing together
• Oxidizing certain metals and gases.

Undesirable dissolved gasses enter water either from the air above the water or as a by-product of some chemicals or biological reaction
the scrubbing process caused by the turbulence of aeration physically removes these gases from solution and allow them to escape into the surrounding air.
The second removal process, oxidation can help remove certain dissolved gases and minerals. Oxidation is the chemical combination of oxygen from the air with un desired in the water (iron and manganese is the common un desired metals) once oxidation these substances come out of solution and become very finely divided suspended material in the water . The suspended material can then be removed by filtration.
The efficiency of the aeration process depends almost entirely on the amount of sur

## Constituents Affected by Aeration

Aeration of water removes troublesome gasses .oxidizes impurities such as iron and manganese so that they can be removed later in treatment, reduces certain types of tastes and odors and introduce oxygen into the water, the constituents commonly effected by

aeration are :-

• Carbon dioxide (CO2)

• Hydrogen sulfide ($H_2S$)

• Methane ($CH_4$)

• Iron (Fe)

• Manganese (Mn)

• Various chemicals causing tastes and odors

• Dissolved oxygen (DO).

## Carbon dioxide:-

• $CO_2$ increases the acidity of water, making it corrosive.

• $CO_2$ tends to keep iron in solution, thus making iron removal more difficult

• $CO_2$ reacts with the lime added to soften water, causing an increase in the cost of softening because of the additional lime that must be added.

## Hydrogen sulfide

Before disinfection of the water chlorine react with H2S which reduce the efficiency of disinfection

$H_2S$ it self is corrosive. It attack corrode pipes as will as concert and metal surfaces in the treatment plants

## Methane

It present in natural in ground water near sits of natural gasses deposits

Its explosive gas, slightly soluble in water and easy removes by aeration

## Iron , Manganese

Iron (in the ferrous, form) and manganese (in the manganous, form) are objectionable for several reasons.

Water containing more than 0.3 mg/ L iron stains most things it contacts yellowish- to reddish-brown. This is especially true of plumbing fixtures. At concentrations of I mg/L

or more, iron gives the water an unpleasant metallic or medicinal taste and can cause turbidity.

Iron can deposit on pipes walls, well screens, and valves. Manganese concentration as low as 0. I mg/ L can produce black stains and cause other problems similar to those caused by iron. Water with both iron and manganese produces stains varying from dark brown to black. Typical consumer complaints are that laundry is stained and that the water is “red” or “dirty.”

Iron and manganese problems can usually be controlled by keeping iron concentrations below 0.3 mg/L and manganese concentrations below 0.05 mg/L. Although there are several ways of removing iron and manganese, aeration followed by filtration is a commonly used technique. Aeration provides the dissolved oxygen (DO) needed to convert the iron and manganese from their soluble forms (ferrous, $Fe_2$ and manganous, $Mn+^{٢}$) to their insoluble forms (ferric, $Fe^{+٣;}$ and manganic, $Mn^{+٣}$), It takes about 0.14 mg/L of DO to remove I mg of iron and 0.27 mg/L of DO to remove I mg of manganese. However, aeration is only the first step. Adequate removal requires a detention time during and after aeration long enough for oxidation to occur completely, followed by filtration (usually pressure filtration) to ensure the removal of the insoluble iron and manganese precipitates.

Tastes and odors. Aeration is effective in removing only those tastes and odors caused by very volatile . materials (materials that turn to vapor easily and have low boiling points). Methane and hydrogen sulfide are two common examples of materials that are removed by aeration. Iron and manganese are examples of easily oxidized minerals.

Many taste and odor problems are thought to be caused by the oils or other by-products that algae produce. Since the oils are much less volatile than tile gases previously mentioned, aeration is only partly effective in removing them. On the other hand, odors caused by certain organic industrial wastes can be quite effectively removed by aeration.

Dissolved oxygen (DO). One effect of aeration is to introduce oxygen into the water where it dissolves to become DO. A certain amount of DO is beneficial in that it increases water’s palatability by removing the “flat” taste. Too much DC) can cause corrosion problems.

Tile amount of oxygen that can remain dissolved in water depends on the water's temperature—the colder the water the higher the possible concentration of DO. Therefore, depending on the oxygen content and temperature of the water being aerated, oxygen might be either dissolved in or released from the water. For example, water in the lower portions of lakes or reservoirs is often very low in DO. Aerating this water will increase its DO content. Sometimes

## Common Operating Problems

As discussed.Aeration easily increases the DO content of water, particularly water that enters the process with a relatively low DO content. In fact, too much DO can be added—this results in SUPERSATURATION, which can cause major operating problems:

- Corrosion of tanks and pipelines
- Floating floc in clarifier
- False clogging of filters (air binding) as the water warms and releases DO,

In addition to super saturation, other operating problems in the aeration process are

- Removal of $H_2S$
- Algae
- Clogged diffusers
- Wasted energy.

## Corrosion

A certain amount of DO is a necessary condition in raw and treated waters. However, DO is also a cause of corrosion. Corrosion can occur whenever water and oxygen come into contact with metallic surfaces. Generally, the higher the DO concentration, the more rapid the corrosion.

The solution to the problem of excessive DO concentration is simply to not over-aeration; unfortunately, there is no definite rule as to what constitutes over aeration. The amount of aeration needed will vary from plant to plant, and may vary seasonally as water quality changes. The correct amount must be determined from experience. However, it is usually

not advisable to supersaturate the water with DO. The saturation levels for DO in water at various temperatures are given in Table

There is two things that can be done to prevent or retard corrosion. First, apply and maintain adequate protective coatings on all exposed metallic surfaces. The application and reapplication of paint and other protective coatings is a vital part of any preventive maintenance program. Second, operate the aeration process so that it provides an adequate but not excessive level of DO. Avoid supersaturating the water with oxygen; that is. keep the DO below the oxygen saturation values in Table 2-I. As a general rule, DO concentrations of 2 to 4 mg/ L should be acceptable, depending on the situation.

## Floating Floc in Clarifier

Excess aeration, particularly at supersaturated levels, can cause serious sedimentation problems. Small bubbles of excess air will come out of solution and attach to the particles of hoc in the clarifier, causing the particles to float. Instead of settling, the hoc particles will float to the surface and flow out of the clarifier with the treated effluent. This destroys the efficiency of the sedimentation process and creates a tremendous load on the filters.

## False Clogging of Filters (Air Binding)

In the same way that bubbles of excess air can attach to floe particles, they can also attach to the media in a filter. This will occur if the water warms as it passes through the filter, releasing the air that was in solution. The process can continue until the spaces between the media particles begin to fill with air bubbles. This is called AIR Binding—it causes the filter to behave as though it were clogged and in need of backwashing. Serious disruption of the filter media can also occur as the air bubbles burst within the bed.

## Hydrogen Sulfide Removal

Hydrogen Sulfide is most efficiently removed by the physical scrubbing action of aeration, not by oxidation. Which of the two removal processes actually takes place depends on the pH of the water. At a pH of 6 or less, hydrogen sulfide exists primarily as

the gas $H_2S$. In this form, it is easily scrubbed away by aeration. However, at pH values of 8 or above, the $H_2S$ gas IONIZES and the ionized form ($HS^-$ and $HS^{-2}$) cannot be removed by aeration.

In fact, the oxygen provided by aeration reacts with the ionized H2S to release the element sulfur, which occurs as a fine colloidal particle and gives water a milky-blue turbidity.

One of the better procedures for the elimination of hydrogen sulfide is to lower pH to 6 or less before beginning aeration. This will favor 112S removal by scrubbing. The pH can be lowered by bubbling $CO_2$ gas into the water, After the H2S is removed, additional aeration will remove any remaining $CO_2$.

## Algae

In warmer climates, the slat and coke tray and positive draft aerators may be located in the open air and exposed to direct sunlight. The wetted surfaces of these aerators create excellent environments for growth of algae and slime. Although it is not practical to completely eliminate these problems, growth rates can be greatly reduced by providing a roof or canopy to shade the aerator from direct sunlight. The growths can also be controlled by periodic treatment with copper sulfate or other suitable chemicals.

## Clogged Diffuser

Air diffusers can become partly clogged, either from dust in the air or from oil. Debris or chemical deposits that can collect around the diffuser opening. Clogging can be minimized by

- Maintaining clean air fillers
- Not over-lubricating blowers
- Preventing backflow of water into diffusers.

## Operational Control Tests

There are three basic control tests involved in operating the aeration process:

• DO

• pH

• Temperature.

The DO test is used to monitor the amount of oxygen being dissolved in the water. Knowing the DO concentration needed and monitoring the amount of DO present will avoid over- or under-aeration.

The pH of water can be used as an indicator of $CO_2$ removal (pH increases as $CO_2$ is removed), and it can also be used to monitor the effective pH range for H2S, iron, and manganese removal. The best p11 range for H2S scrubbing is 6 or less, whereas iron and manganese arc best treated in a pH range of 8 to 9.

The amount of oxygen which will dissolve in water (the saturation concentration) varies depending on water temperature, increasing as the water temperature drops. As water temperature drops, the operator must adjust the aeration process to maintain the correct level of DO.

Additional control tests for the following are used to measure the efficient or effective the aeration process is in removing troublesome constituents:

• Iron

. $CO_2$

• Manganese

• Taste and odor.

Test frequency depends both on the variability of raw-water quality and on the characteristic measured. For example, well water tends to be very constant in quality and

very slow to change, so daily testing may not be necessary.
Surface-water quality is subject to more frequent change. Temperature, DO, pH, $CO_2$. and tastes and odors can vary daily. Therefore, sampling and testing frequencies should be set up so that all significant changes can be detected and monitored.
In addition to these specific control tests, it is advisable to make frequent visual inspections of the process. Clogged diffusers, for example, are common problems that are most easily identified visually.

## Safety and Aeration

Hydrogen sulfide is a poisonous gas, which is colorless, flammable, and explosive. The gas kills by paralyzing the respiratory center; a few minutes of exposure to a concentration of $H_2S$ as low as 0. I percent by volume in air is fatal. Higher concentrations, above 4.3 percent. are explosive. A very dangerous characteristic of $H_2S$ is that its foul, rotten-egg odor is not evident at the dangerous concentrations, because such concentrations paralyze the sense of smell (the olfactory nerves).
METHANE, or natural gas, is a colorless, odorless, and tasteless gas, which is flammable and highly explosive. If the scrubbed gases containing methane are allowed to accumulate in a confined space, an explosion or fire can result. Methane can also kill by simple asphyxiation (suffocation); and since the gas has no color, taste, or odor, the victim can be completely unaware of its presence. Because methane is odorless, the natural gas piped to homes, industry, and commerce is treated with a strong odor-causing chemical, allowing leaks and accidental discharges to he easily detected.
The safest way to deal with hydrogen sulfide and methane gases is to make sure that the aeration process is well ventilated and that there are no locations within the process where gases could accumulate. Methane is lighter than air and will accumulate at the top of an enclosure. Hydrogen sulfide is heavier than air and will collect at the bottom of confined areas, Ideally, aerators should be located in an open area with good ventilation. Where aerators must be enclosed due to climatic conditions, the enclosure must he well ventilated at all tithes.

# COAGULATION/FLOCCULATION

The coagulation/flocculation processes help to remove non settable solids. COAGULATION involves the feeding and rapid mixing of one or more chemical COAGULANTS into the water, there by begging the formation of particles called FLOC. FLOCCULATION, which party overlaps the coagulation process, requires the gentle mixing of the water and coagulants for a period of time in order to form larger, heavier, more settable floc particles. These floc particles are a combination of the coagulant and the particles in the water; they are larger and heavier than the original particles and considerably easier to remove in later treatment processes such as sedimentation and filtration.

This section describes the principles involved in coagau1ation and flocculation. the common coagulants and coagulant aids in use and the common operating problems associated with the process. The module concludes with a discussion of the safe handling and use of coagulant chemicals, particularly alum.

## After you have completed this section, you should be able to

• Describe the principles involved in coagulation and flocculation.

• List the common chemical coagulants, their advantages and disadvantages.

• Define the following:

-Turbidity

- Coagulation

-Coagulant aid

- Flash mixing

- Flocculation

• explain why .good mixing is important.

• Identify two common operating problems associated with the coagulation/ flocculation process, and describe how these problems might be resolved.

• List reasons for using coagulant aids.

• List and briefly describe the purpose of each test used to monitor the effectiveness of

the coagulation/ flocculation process.

• Describe safe handling practices for coagulant chemicals.

**Description of Coagulation/Flocculation**

The coagulation/flocculation process is necessary in water treatment manly because of NONSETTLEABLE SOLIDS, particles too sma11 to effectively removed .by later treatment processes such as sedimentation and filtration These non settable solids can be converted in o larger and heavier settable solids by physical—chemical changes brought about by adding and mixing chemical coagulants into the raw water. The settleable solids can then be removed by the sedimentation and filtration processes.

**Basic Principles**

In order to understand how the coagulation/flocculation process works, it is important to know the natural of non settleable particles

- Particle size
- Natural forces between particles.

**Particle Size:-**

Untreated, natural water contains three types of nonsettleable solids, from largest to smallest

- Suspended
- Colloidal
- Dissolved solids.

**Table 3-1. Size Range of Solids**

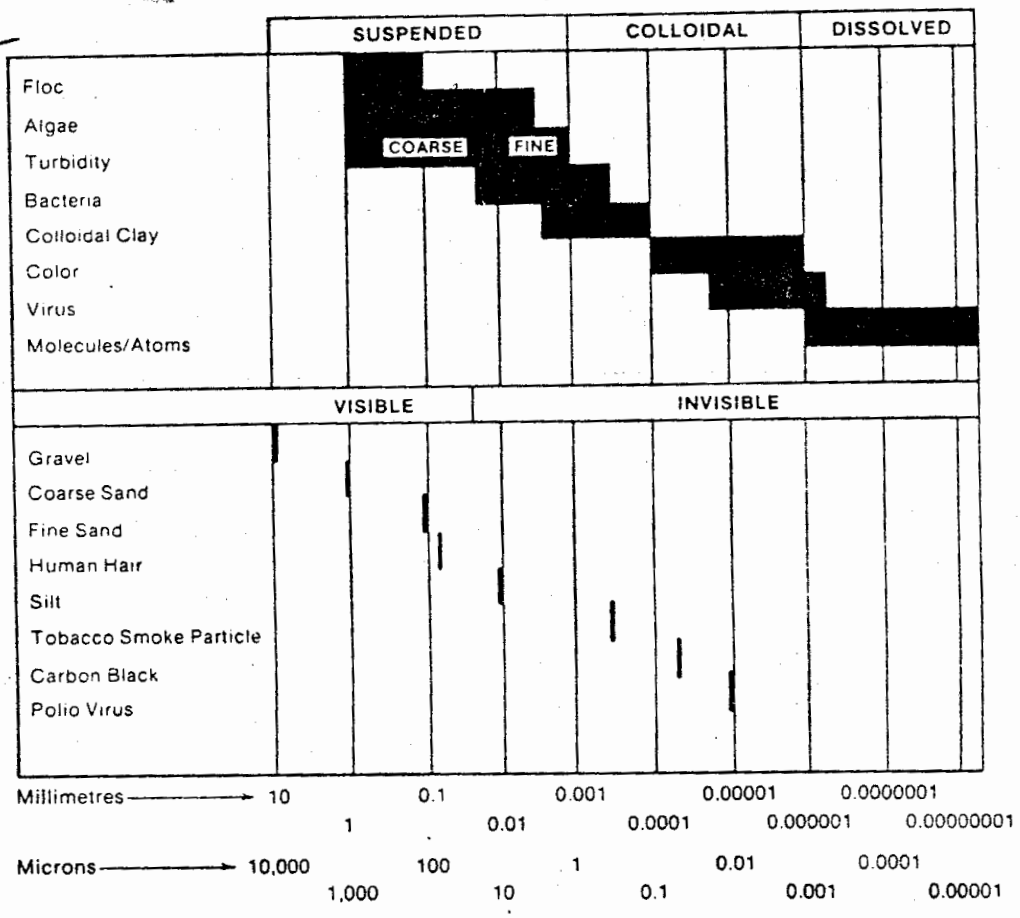



**Table 3-2. Natural Settling Rates for Small Particles**

| *Particle Diameter* mm | *Representative Particle* | *Time Required to Settle in 1 ft (0.3 m) Depth* |
|---|---|---|
| | | Settleable |
| 10 | Gravel | 0.3 sec |
| 1 | Coarse sand | 3 sec |
| 0.1 | Fine sand | 38 sec |
| 0.01 | Silt | 33 min |
| | | Considered nonsettleable |
| 0.001 | Bacteria | 55 hour |
| 0.0001 | Color | 230 day |
| 0.00001 | Colloidal particles | 6.3 year |
| 0.000001 | Colloidal particles | 63-year minimum |

Source: *Water Quality and Treatment.* AWWA, Denver, Colo. (3rd ed., 1971)

**Suspended SOLIDS**

are particles carried along (held in suspension) by the natural action of flowing water. The smallest suspended solids (less than 0.01 mm )

**Colloidal Solids**

Do not settle quickly for purposes of water treatment are called non settleable. Large suspended solids (greater than 0.01 mm) are referred to as settable solids since the will settle unaided to the bottom of a container or sedimentation basin within four hours. Examples of COLLOIDAL SOLIDS include fine silts, bacteria, color-causing particles and virus. These colloids do not settle settle in a reasonable period of time although individual colloidal solids can not seen by naked eye, their combined effect is often seen as color or turbidity in water. These particle are small enough to pass . through later treatment process if not properly coagulated and flocculated.

**DISSOLVED SOLIDS**

are any organic or inorganic matter—such as salts, chemicals of plants or animals origin or gasses - that is dissolved in water. A dissolved solid is the size of a molecule and is invisible to the naked eye . Most of the trace metals and organic chemicals found in water are dissolved. They are non settleable and can cause public health and other problem such as taste , color or odor . unless converted to a precipitate by chemicals or physical means, they cannot removed from the water .

One of the main reasons the nonsettleab1particle resist settling is their small size. Consider this example: A particle of coarse , sand hat is the shape of a cube, 1 mm on a side, could be expected to settle quickly, dropping about (0.3 m) every second .Now suppose the particle of ground into many smaller particles, each a cube 0.000001mm on a side. the weight of all grinding the same as the original grain of coarse sand, but the exposed surface area has increased from $1mm^2$ to 6 $m^2$, the area of the head of a large pin, to 6 $m^2$ .The increase in surface area causes a tremendous increase in the forces (often called drag forces) that resist settling. Instead take 3 sec. it would now take about

60 years for these tiny particles to settle the same 0.3 m depth the increasing in the settling time due to decreasing in the particle size. To promote settling, the particles must be brought together to form larger particles
that settle easily and quickly.

**Natural forces effect on settling particles**

1. Particles in water usually carry –ve charge. Just as like pole of magnet repel each other . that there are repelling force between any two particles of like charge in water . that natural electrical repelling force is called zeta potential . this force strong enough to hold the very small colloidal particles apart and keep them in suspension

2. The van der waals force exist between all particles in natural and tend to pull any to particles together . this attraction force acts opposite to zeta potential .As long as zeta potential is stronger than van der waals force the particles will stay in suspension

**Effect of coagulation / flocculation .**

This process neutralizes or reduce zeta potential of non settable solids so the Van Der Waals force force for attraction can begin pulling particles together. The Non settable particles are then abele to gather into small group of microfloc . These particles though larger than the original colloids, are held together weakly. Individual microfloc are invisible to the naked eye and are still non settable . The gentile stirring action created by flocculation brings the floc particles to gather to for large and relatively heavy floc particles which can easily be settle or filtration

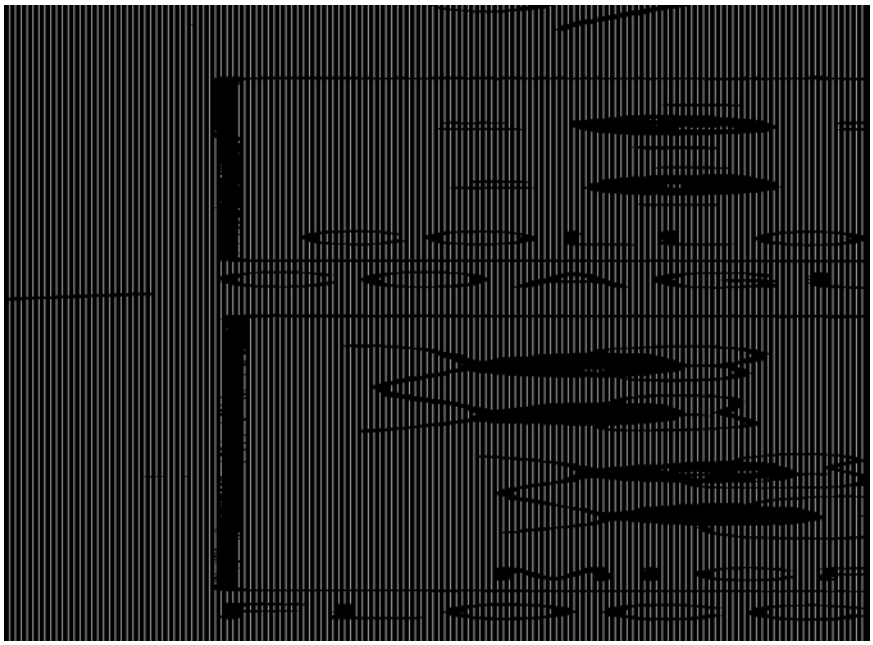
* The most commonly used coagulants in water treatment are ALUMINUM SULFATE (usually called ALUM), $A1_2(S0_4)_3$ and FERRIC SULFATE, $Fe_2(S0_4)_3$. When either of two coagulants dissolve and ionize in water form the trivalent ions of aluminum ion ($Al^{+3}$or $Fe^{+3}$ )

1. Alum added to the raw water reacts with the alkalinity naturally present, to form jelly like floc particles of aluminum hydroxide, $Al(OH)_3$. (A certain level of alkalinity is necessary for the reaction to occur.

2. The positively charged trivalent aluminum ion neutralize the negatively charge particles of color or turbidity . This occurs within one or two second after the chemical
   is added to the water ..which is whey rapid thorough mixing is critical to good coagulation . this neutralization of
   electrical charges marks the beginning of coagulation.
   At this time, the particles begin to come together to form larger particles.

3. The floc that is first formed is made up of small particles (microfloc) which still positive charge from the coagulant added. They continue to neutralize negatively charged particles, and in this way become neutral particles.

4. Finley the microfloc particles begin to collide and stick together (agglomerate) to form larger is process is called flocculation



* There are many physical and chemical factors that affect the success of a particular coagulant, include pH , Turbidity ,Alkalinity and temperature o the water.

For example , alum work best in a pH range of about 4.5 to 8.5 if alum is used outside this range, the floc either does not form completely or it forms and then is dissolved due to the high or low pH operation may not always match these results. Actual plant performance must be monitored for:

- Adequate flash mixer
- gentile flocculation
- adequate flocculation time
- settle and filtration water Quality

Successful coagulation is based on rapid and complete mixing .Through coagulation occurs in less than a second the chamber may provide up to 60sec of detention time. Mixing should be turbulent enough so that the coagulant is throughout the coagulant chamber. Some experts maintain that during the first one-tenth of a second the coagulation must be thoroughly mixed with every drop of water to start an efficient floc or, from that point, the efficiency of the entire process declines. Most improper coagulant usage shows its effect at this moment, and the problem can usually be corrected by altering either the point of application or method of mixing

* The following indicators is due inadequate mixing

1. Very small flock ( pin point floc)
2. Fish eye
3. High turbidity in settle water
4. too frequently filtration backwash

If any of these conditions appear, the operator should begin to investigate the adequacy of the flash mixing operation, the gentleness of flocculation, and the length of flocculation time

Proper flocculation requires long, gentle mixing. be high enough to bring coagulated particles constantly into contact with each other, but not so high as to break those particles already flocculated . For this reason, flocculation basins and equipment are usually Design to provide High mixing speeds

## Common operation problems

There arc primarily three operating problems that can interfere with successful coagulation/flocculation:

• Low water temperature
• Weak floc
• Slow floc formation.

**low water temperature** :-

. Low water temperatures, particularly temperatures approaching freezing , interfere with the coagulation/flocculation process. As water temper. Decreases, the viscosity of the water increases. This decrease the rate of floc settling

The colder temperature also slows chemical reaction rates, the best pH for the chemical reaction drops as temperature drops.

Cold water has a tendency to penetrate through the filters, indicating that floc strength has decrease

This problem can solve by using any of these three techniques:

• Operate the coagulation process as nearly as possible to the best pH value for that temperature water.

• Increase coagulant dosage. This increases the number of particles available to collide a also reduces the effect of changes in drop in temperature

• Add weight agent such as clays, to increase floc density

**Weak floc**

The weak flock dose not adhere on filter media but it break down and produce poor quality effluent

This condition due to inadequate rapid mixing or inadequate slow mixing in flocculation step

Use Jar test to determine the best condition for mixing (rapid , slow )

**Slow floc formation**

This problem occur at low turbidity raw water

We can solve this problem by return previously settling sludge

Or artificially by add weighting agent

## Operational Control Tests

There are five tests that can help in controlling the coagulation/flocculation process:

- Jar test
- pH
- Turbidity
- Filtrability
- Zeta potential
- Jar Test

**Jar test**

The jar test helps determine the chemical or chemical combination and the dosage that will produce the best floc. testing should not be limtid to the chemical or chemicals that the plant has always used. A wide variety of coagulants and coagulant aids should be

tried. Suppliers will be able to furnish test samples. Periodically, new chemicals that have come on the market should be tested. This testing can lead to the use of new coagulants that are as effective as currently used chemicals, yet are less costly or produce less sludge.

**pH**

Coagulants and coagulant aids operate best within a particular range of pH. Knowing the pH of the water helps an operator select the proper coagulant or correctly adjust pH prior to coagulation

**Turbidity.**

In the past, the clarity of the settled jar test samples was often evaluated by eye and described with terms such as “poor,” “good,” and “very good.” Subjectively , and the errors that it can cause, can be avoided by measuring the
turbidity of the settled jar test samples

**Filter ability**

measures the amount of water filtered in a given time. It is usually made by applying small portion lcoagulated/flocculated water before sedimentation through pilot filter has the same media of actual filter

**Zeta potential**

the Zeta potential test has been successfully to help determination the best pH , and dosage of cation coagulant $AL^{+3}$

Table 3-5. Degree of Coagulation Within Different Zeta Potential Ranges

| *Average Zeta Potential* | *Degree of Coagulation* |
|---|---|
| +3 to 0 | Maximum |
| −1 to −4 | Excellent |
| −5 to −10 | Fair |
| −11 to −20 | Poor |
| −21 to −30 | Virtually none |

# Jar Tests

## PURPOSE OF TEST

Jar tests show the nature and extent of the chemical treatment that will optimize the quality of water that leaves the plant. Many chemicals added to a water supply can be evaluated on a laboratory scale by means of a jar test. Among the most important chemicals are coagulants, coagulant aids, alkaline compounds, softening chemicals, and powdered activated carbon for taste and odor removal and adsorption of organics. When coagulants are added, the jar test may be called a coagulation or flocculation test.

Surface waters generally contain suspended particles that cause turbidity.
These particles vary in size and amount. Turbidity can be removed by adding a coagulant such as alum, followed briefly by rapid mixing, then a longer period of slow mixing (flocculation), and finally a period of settling. The laboratory coagulation test that uses jar testing equipment is an attempt to imitate the mixing, flocculation, and settling conditions in the full-scale treatment plant.

Select a series of doses so the first jar represents under treatment and the last jar represents over treatment. This approach indicates the correct coagulant dosage for the plant when varying amounts of turbidity, color, or other factors dictate a change in the coagulant dose. When this information is fine-tuned and implemented, it helps the plant produce a clear, colorless water.

Jar tests can also be used to compare the performance of various types of coagulants and coagulant aids before they are used in the full-scale treatment plant. Separate tests for each coagulant can be performed on the same water. Floc formation, turbidity removal, color removal, pH, and cost per thousand gallons can then be compared for the optimal dose of each coagulant.

## LIST OF SIMPLIFIED METHODS

Jar tests are conducted using commercially available stirring machines that have three to six paddles. Tests can be conducted to determine correct dosages and treatment regimes for coagulation, lime or soda ash softening, and activated carbon

addition. After chemical treatment, samples are collected and analyzed for turbidity, pH, hardness, odor, organic compounds, or any other parameter of interest. See other chapters of this manual for descriptions of these tests.

## SIMPLIFIED PROCEDURES

# General Jar Test Information

**Warnings/Cautions.**

Even the smallest detail may have an important influence on the result of a jar test. Therefore, all samples in a series of tests should be handled as similarly as possible.

The purpose of the test determines the necessary experimental conditions, e.g., stirring speed, the length of the flash mix, flocculation, and settling intervals. The best results are achieved when jar test conditions closely match those used in the full-scale plant for each of these controllable parameters.

Temperature plays an important role in coagulation, so source water samples should be collected only after all other preparations have been made. Source water may be stored for longer periods of time if the water can be kept at the same temperature as the water being treated, i.e., submerged in a water bath.

## Apparatus.

• a stirring machine with three to six paddles that can operate at variable speeds (0 to 270 rpm)

• a floc illuminator for observing floc formation (optional)

• special, square, 1 or 2L jars with side sample ports, sometimes called gator jars (conventional beakers or circular jars with baffles can be used, but sample collection is more difficult)

• a large plastic carboy for collecting source water

• a thermometer

• top loading balance

• 1-L volumetric flasks

• disposable plastic syringes, such as B-D Plastipak syringes or equivalent, for dosing samples rapidly with coagulants and other solutions (most common sizes are 3, 5, 10, and 20mL volumes)

• 100-mL pipettes for obtaining samples if gator jars with side ports are not used

• sample containers, such as small beakers, to collect treated jar test samples

• analytical equipment required for tests such as turbidity and hardness

## **Reagents**.

**.**Dosing solutions. Suspensions or solutions prepared from the stock materials actually used in the plant.

**.**Distilled water. Boil distilled water for preparing lime suspensions for 15 min to expel carbon dioxide.

## **Procedure.**

Always select a series of doses for the chemical being tested so the first jar represents undertreatment and the last represents overtreatment. When a proper series is set up, the succession of jars will show poor, fair, good, and excellent results at the end of the test. In the case of coagulation, overtreatment may also yield poor results. Repeat the jar test once or twice to find the proper series of doses for the desired results. The objective is to determine the lowest (and therefore the least expensive) dose that will produce the desired water quality.

Rapid mix and flocculation mixing intensities and stirring times should imitate those used in the full-scale plant, when possible. Because of differences in energy transfer to the water in the jar by the stirrer and in the water in the treatment plant by the rapid mix and flocculation impellers, detention time may differ between the

jar and treatment plant basins. Scale calculations may be required.

Plant operating conditions may also dictate a change in the laboratory stirring times. For example, the flocculation period in a plant that operates at 5 mgd during winter may be 40 min, but a summer pumping rate of 10 mgd reduces the flocculation period to 20 min. Make similar downward adjustments when a plant has parallel flocculation basins but one is taken out of service, thereby reducing the effective flocculation period.

## Coagulation Jar Test

**Warnings/Cautions.**

Refer to the general jar test information discussed earlier in this chapter.

Many coagulants are irritants to skin, and contact with concentrated solutions should be avoided. Wash thoroughly with water if contact occurs. Read Material Safety Data Sheets for specific information.

Dilute coagulant solutions have a very limited shelf life. Follow the instructions listed below for each coagulant type.

**Apparatus.**

- 1-L and 250-mL beakers
- a 500-mL volumetric flask
- a hand-held electric blender

**Reagents.**

Many chemicals and other materials are used to coagulate turbid or colored waters. Primary coagulants are generally inorganic and used alone or in conjunction with coagulant aids.

Alkalinity, which is sometimes required to produce the desired hydrous oxide

floc, can be added in the form of calcium hydroxide (hydrated or slaked lime), calcium oxide (quicklime or unslaked lime), or soda ash.

Clay suspensions that consist of clay, kaolin, bentonite, or stone dust may be added to improve the coagulation of low-turbidity water from which it is difficult to form floc nuclei. Inorganic coagulants such as aluminum sulfate (alum), ferric chloride, ferric sulfate, and polyaluminum chloride.

1. These coagulants should be prepared daily. If possible, obtain concentrated coagulant from the supply actually used in the plant.

2. Weigh 10 g of the material. If the coagulant is a liquid, this is most easily done by drawing it into a plastic syringe and weighing it on the balance, after first weighing the syringe empty or taring the balance.

3. Dissolve in 500 mL distilled water in a 1-L volumetric flask. Dilute to 1 L with distilled water. Mix thoroughly before each use. Each 1 mL of the resulting solution represents 5 mg/L as coagulant product in a 2-L water sample.

NOTE: Only a certain portion of the coagulant product is actually the working coagulant chemical. For example, alum may be 48.9 percent $Al_2(SO4)_3$, thus 1 mL of the resulting solution prepared above would represent 2.4 mg/L Al2(SO4)3, in a 2-L water sample. Therefore, be sure to account for the percentage of active ingredient when calculating dosages for inorganic coagulants.

***Liquid polymer stock solution.***

Polymers are organic materials that can be used as coagulant aids. Polymers are classified as cationic (positively charged), anionic (negatively charged), or nonionic (no charge).

**Prepare a 1 percent weight to volume solution.**

1. Use a volumetric flask to measure 500 mL distilled water. Place the water into a 1-L beaker, and then use a syringe to withdraw 5 mL water. This

leaves 495 mL water in the beaker.

2. Place a 5-mL plastic syringe on a top-loading balance and record the weight or tare the balance.

3. Shake the neat polymer until thoroughly mixed. Fill the syringe with 5 g neat polymer (about 4.8–4.9 mL). Weigh the syringe on the balance. Add or remove polymer from the syringe until the correct weight is reached.

4. Place the hand-held electric blender into the dilution water in the 1 L beaker. Start the blender, place the syringe tip below the water surface in an area of high turbulence and slowly empty the polymer from the syringe. Be careful not to inject the polymer directly into the beaker walls or blender surfaces.

5. Continue to mix for 30–60 s. Let this stock solution age at least 15 min before preparing the 0.1 percent working solution.

6. Prepare this solution every 48 h.

**Prepare a 0.1 percent working solution.**

1. Place 180 mL distilled water in a 250-mL beaker. Fill a 20-mL syringe with 20 mL 1 percent stock polymer solution. Be sure there are no air bubbles present.

2. Place the hand-held blender into the dilution water in the beaker. Start the blender, place the syringe below the water surface in an area of high turbulence, and then slowly empty the polymer from the syringe. Be careful not to inject the polymer directly into the beaker walls or blender surfaces.

3. Prepare fresh working solutions daily, preferably less than 2 h before use.

4. Each 1 mL of the resulting working solution represents 0.5 mg/L in a 2-L water sample.

## Lime or Soda Ash Solutions.

1. Distilled water used to prepare lime suspensions should be boiled for 15 min to expel the carbon dioxide. Cool to room temperature shortly before adding lime.

2. Weigh 10 g of solid material on a top-loading balance. Suspend in 1 L

distilled water in a volumetric flask. Mix thoroughly immediately before each use.

3. Each 1 mL dosing solution represents 5 mg/L in a 2-L water sample.

4. Prepare daily.

**Procedure.**

1. Clean the gator jars using detergent and a scrub brush. Rinse thoroughly with tap water.

2. Clean the stirring machine paddles with a damp cloth.

3. Collect the water to be treated in a plastic carboy. Unless the water can be stored at ambient temperature, begin the jar test procedure promptly so the water temperature does not change significantly.

4. Mix the sample water thoroughly and fill all the gator jars half full. Remix the sample water. In reverse order, fill the gator jars to the fill line.

5. Place the paddles into the jars and tighten the thumb screws.

6. If using a clay suspension, add it to each jar with a wide-tip pipet, a plastic syringe, or a graduated cylinder.

7. Fill plastic syringes with the correct amounts of coagulant or coagulant aid solution to be added to each jar. Ensure there are no air bubbles, especially near the tip of the syringe. If bubbles are present, empty and refill the syringe. Fill one syringe for each jar.

8. Begin rapid mixing by setting the machine to the desired paddle speed (rpm value). In a 2-L square gator jar a mixing speed of 270 rpm corresponds to a mixing intensity of about 500 s–1.

9. Inject the coagulant solutions into the jars below the surface of the water near the paddles. Inject all the jars at the same time for each type of coagulant solution added.

10. Add chemicals in the order they would be added in the plant and mix each one for the same time period mixing would occur in the plant.

11. If desired, use plastic syringes to add lime or soda ash to add alkalinity to

waters deficient in natural alkalinity. See Table for the amounts of lime or soda ash needed to neutralize various coagulants.

12. Lower the paddle speed to simulate the flocculation mixing process. If possible, mix the water at the same mixing gradient and for the same time period as occurs in the plant. Figure 16-1 shows the correlation between rpms in the gator jar and the mixing intensity within the jar.

13. Observe each jar for the appearance of “pinpoint” floc and note its time and order.

14. At the end of the flocculation period, stop the stirring machine, remove the paddles, and allow the floc to settle.

15. Observe the floc as it settles and describe the results. A hazy sample indicates poor coagulation. Properly coagulated water contains well-formed floc particles, and the liquid between the particles is clear.

16. Collect samples after 5, 10, 15, and 30 min or as desired. Collect samples by opening the clamp on the sample tube on the side of the gator jar, flushing 20 mL to waste, and collecting the necessary amount of sample into a sample cup. Do not clamp the tube shut between the flush and the sample collection, and make sure the tube is 100 percent open. If beakers are being used instead of gator jars, obtain the sample carefully with a 100-mL pipet.

17. Analyze the samples for turbidity, pH, and any other parameters of interest using the methods described in other sections of this manual.

Table 16-1 Chemical addition of alkalinity for jar testing

| Chemical | mg/L required to react with 1 mg/L of a given coagulant | | | |
|---|---|---|---|---|
| | Total Alkalinity (as $CaCO_3$) | Soda Ash (as $Na_2CO_3$) | Lime [as $Ca(OH)_2$] | Lime (as CaO) |
| Alum | 0.50 | 0.54 | 0.39 | 0.33 |
| Ferric Sulfate | 0.75 | 0.82 | 0.59 | 0.50 |
| Ferric Chloride | 0.92 | 1.0 | 0.72 | 0.61 |

NOTE: These dosages provide a starting point. Actual dosages are determined by the jar test.

# SEDIMENTATION

Sand, grit, chemical precipitates, pollutants, floc, and other settleable solids are kept suspended in water by the velocity and Turbulence (agitation and irregular flow) of the flowing water. Sedimentation removes these solids in order to decrease loading on filtration and other treatment processes . Basins designed for efficient sedimentation allow the water to flow very slowly. Because velocity is low and turbulence is decreased, the suspended solids settle to the bottom of the basin. Sludge a residue of solids and water, accumulates at the bottom of the basin and is pumped out of the basin for DEWATER (removing water from sludge) and eventual disposal or reuse.

**After completing this module you should be able to**

- Describe the role of sedimentation in water treatment.
- List the key parts of settling basins.
- identify and describe the function of the four zones of sedimentation.
- List important factors in the improper operation of settling basins.
- Identify the records that should he maintained on the sedimentation process.
- Describe the common operating problems associated with settling basins.
- State which tests should be run to properly monitor sedimentation performance.
- Describe the safety considerations associated with the sedimentation process

## Description of Sedimentation

Sedimentation, sometimes called CLARIFICATION, is the removal settleable solids in water. The solids removed include sand, silt, grit chemical and other settleable solids. The sedimentation process takes place in a SEDIMENTATION BASIN or SETTLING BASIN. Sedimentation occurs after coagulation / flocculation and before filtration in a conventional water treatment plant.

## Zones in a Sedimentation Basin

Regardless of the type of basin used, four zones exist, each with its own function. These zones,

1. Inlet zone. The INLET ZONE decreases the velocity of the incoming water and distributes the flow evenly a6F1Tati’’’

2. Settling zone. The settling Zone provide the calm (quiescent) area necessary for the suspended material to settle.

3. Outlet zone. The OUTLET ZONE provides smooth transition from the settling zone to the effluent flow area. It is important that Currents or eddies that could stir up any settled solids and carry them into the effluent do not develop in this zone.

4. Sludge zone. The SLUDGE ZONE receives the settling sludge and keeps them separate from other articles in the settling zone.

These zones are not as well defined as in the illustration. in actual operation, the settling zone is directly affected by the other three zones, causing less efficient settling. The greater the effect on the settling Zone that due to poor design or operating practice, the worse .the effluent quality.

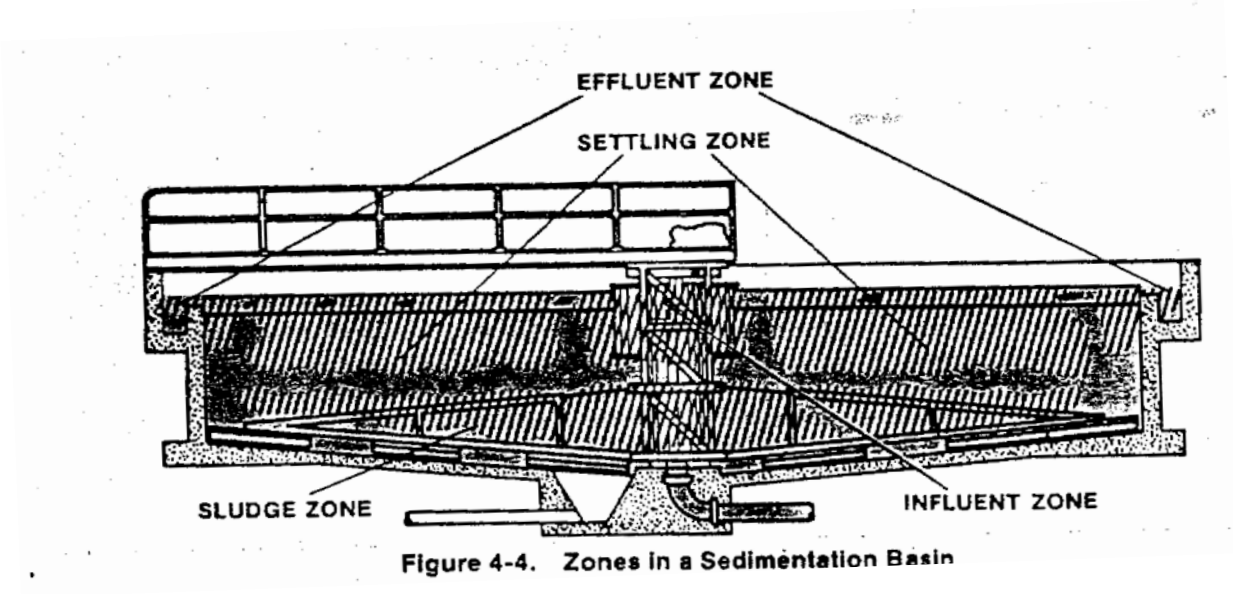



Figure 4-4. Zones in a Sedimentation Basin

## Operating Problems

*Poorly formed floc and short circuiting are the two most common problems in sedimentation.

- Both degrade the settled-water quality and increase turbidity loading on the filters.

  Poorly formed floc, characterized by small or loosely held particles, does not settle properly and is carried out of the settling basin. This condition is the result of poor coagulation and flocculation. Inadequate rapid mixing, improper coagulants or dosages, and improper flocculation are the primary causes of poor coagulation/flocculation. JAR TESTS can provide the information necessary to find the specific problem. The solution may be to increase mixing energy, use a coagulant aid, or install baffles in the flocculation basin to prevent short circuiting.

- Short circuiting—water bypassing the normal flow path through the basin and reaching the outlet in less than the normal detention time—occurs to some extent in every basin. It can be a serious problem in some installations, causing to be carried out of the basin due to shortened sedimentation time. The major cause of short circuiting is poor inlet baffling. If the influent enters the basin and hits a solid baffle, .strong currents and short circuits will result. A perforated baffle can be very successful in providing effective distribution at the inlet without causing troublesome currents .If short circuiting is suspected, TRACER STUDIES using salt or a dye tracer to determine the actual detention time can be conducted.

- DENSITY CURRENTS can also be a problem. They occur because the influent contains more suspended solids, which cause it to have a greater density than the water in the basin. They can also occur when the influent is colder than the water in the basin. In either case, the denser influent sinks to the bottom of the basin, where it can create up swells of sludge and short circuits. If this is a problem, an

engineering study should be conducted to determine the best solution. Many times the problem can be lessened by modifying the effluent weirs.

Wind can cause currents in open basins, which will result in short circuiting if the basins are not protected. A barrier should be constructed around uncovered basins in order to lessen the wind's effect and keep debris out of the water

Another problem occurring in open basins located outside is algae and slime growth on the walls. These growths can cause tastes and odors. If the algae and slime detach from the walls, they can clog weirs or filters. They also create very slick, hazardous surfaces. The growths can be controlled by coating the walls

## Control Tests

The major test used to indicate proper sedimentation is the test for turbidity. The turbidity of samples taken from the raw water and outlet of each basin should be tested at least three times a day—more frequently if water quality is changing rapidly. By comparing these turbidities, an indication of the efficiency of the sedimentation process can be gained. For example, if the raw-water turbidity is 50 NTU and effluent turbidity is 40 NTU, very little turbidity is being removed. This indicates that better coagulation/flocculation is needed or that short circuiting is occurring. Visual examination of water samples from the effluent can also indicate if floe is being carried over onto the filters.

Turbidity of the settled water should be kept below 10 NTU. if turbidity is above 10 NTU, the coagulation/flocculation and sedimentation processes should be checked and their operation improved..

As mentioned in Module 3, Coagulation/ Flocculation, the temperature of the raw water should be measured and recorded at least daily. As water gets colder, it becomes more viscous, thus presenting more resistance to the settling particles. To compensate for this, the surface overflow rates may have to be reduced.

## Safety and Sedimentation

Open sedimentation basins should be equipped with guardrails that will prevent falls into the basins. Walkways and bridges connected to basins should also have guardrails. Life rings or poles should be kept near the basins for rescue purposes.

Particular care must be taken when rain, ice, or snow is on walkways, since they become very slippery. Caution is also needed when cleaning or inspecting drained basins, since growths of aquatic organisms and sludge deposits can make the surfaces slippery.

Moving parts of all machinery should be equipped with guards to prevent the machinery from catching legs, fingers, or clothing. Guards must be left in place while the machinery is operating.

# SOFTENING

Water contains various amounts of dissolved minerals. Some of these minerals, calcium and magnesium in particular, cause HARDNESS. Consumers frequently complain about the problems caused by hard water, such as the formation of scale in cooking utensils and hot-water heaters. The softening. process removes the hardness-causing minerals from water. The two most common water softening techniques are the LIME /SODA ASH and the ION-EXCHANGE PROCESS.

After completing this section you should be able to:

• Describe the purpose of softening water.
• Name the minerals causing water hard ness.
• Identify the various types of water hardness.
• List the methods of softening water.
• Describe the lime-soda ash softening process.
• List the major operating variables of each process.
• Identify typical operating problems and solutions.
• Describe the commonly used laboratory control tests.

## Chemistry and Effects of Hard and Soft Water

Hard water is caused by soluble, DIVALENT, metallic CATIONS (positive ions having a valence of 2) The two main cations that cause water hardness are calcium (Ca) and magnesium (Mg). Although strontium, aluminum, barium, iron, manganese, and zinc also cause hardness in water, they are not usually present in large enough concentrations to contribute significantly to total hardness.

Water hardness varies considerably in different geographic areas due to the different geologic formations in each area and the length of time the water has been in contact with these formations. Calcium is dissolved as water passes over and through limestone

deposits. Magnesium is dissolved as water passes over and through dolomite and other magnesium- bearing minerals. Because ground water is in contact with these geologic.

Formations for a longer period of time than surface water. Ground water is normally harder than surface water.

Expressing hardness concentrations. Water hardness is generally expressed as a concentration of calcium CARBONAT (in terms of mg/ I. as $CaCO_3$). the degree of hardness consumers consider objectionable will vary. Depending on the water and on the degree of ,hardness. to which consumers have become accustomed. Consumers who are accustomed to soft water of 20 mg/ L hardness as $CaCO_3$, for example, might consider 100mg/ L excessive; consumers who are accustomed to very hard water might consider 100 mg/ L quite acceptable. Table shows a comparative classification for softness and hardness in water.

**Types of hardness. Hardness can be categorized by either of two methods:**

• Calcium and magnesium hardness

• Carbonate and noncarbonate hardness.

Calcium and magnesium hardness is identified by the minerals involved Hardness caused by calcium is called Calcium HARDNLSS. regardless of the salts associated with it, which include calcium sulfate ($CaSO_4$). calcium chloride ($CaCl_2$), and others. Likewise, hardness caused by magnesium is called MAGNESIUM HARDNESS. Since calcium and magnesium are normally the only significant minerals that cause hardness, it is generally assumed that total Hardness = Calcium Hardness + Magnesium Hardness.

Carbonate and noncarbonate hardness are identified both by the bicarbonate salts of calcium and by the normal salts of calcium and magnesium involved in causing the water hardness. CAR RONATE Hardness is primarily caused by the bicarbonate salts of Calcium and magnesium - that s. calcium bicarbonate $[Ca(HCO_3)_2]$ and magnesium bicarbonate $[Mg(HCO_3)_2]$ .

Ca, Mg combined with carbonate ($C0_3$) also contribute to carbonate hardness. Noncarbonate Hardness is measure of calcium and magnesium salts other than carbonate and bicarbonate salts . that is, calcium sulfate, calcium chloride . magnesium sulfate($MgSO_4$), and magnesium chloride ($MgCl_2$). Although it is a very rare condition, calcium and magnesium combined with nitrate ($NO_3$) may also contribute to noncarbonate hardness. For/carbonate and noncarbonate hardness,

Total Hardness = Carbonate Hardness + Noncarbonate Hardness

When hard water is boiled, carbon dioxide ($CO_2$) is driven off. Bicarbonate salts of calcium and magnesium then precipitate (settle) out of the water to form calcium and magnesium carbonate PRECIPITATES. These hardness-causing precipitates form the familiar chalky deposits on teapots and other containers. Because it can be removed by heating, carbonate hardness is sometimes called TEMPORARY HARDNESS. Since noncarbonate hardness cannot be removed or precipitated by prolonged boiling, it is sometimes called PERMANENT HARDNESS.

Objections to hard water. Hard water forms scale, usually calcium carbonate, which causes a variety of problems. Left to dry on the surface of glassware and plumbing fixtures (shower doors, faucets, and sink tops), hard water leaves unsightly white scale, called water spots. Scale that forms on tile inside of water pipes eventually reduces the pipes' carrying capacity. Scale that forms within appliances and water meters causes wear on moving parts. When hard water is heated, scale forms much faster. This creates problems inside boilers, water heaters, and hot-water lines. A coating of 0.04 (1 mm) of scale on the heating surfaces of a water heater creates an insulation effect that will increase heating costs by about 10 percent.

The historical objection to hardness has been its effect on soap. Hardness ions form precipitates with soap. These precipitates are unsightly and result in laundering or washing problems. The bathtub ring is a familiar example of the precipitate formed by hard water and soap. To counteract these problems, synthetic detergents have been developed and are used almost exclusively for washing clothes and dishes. Synthetic detergents have additives known as SEQUESTERING AGENTS that "tie-up" the hardness ions so they cannot form the troublesome precipitates. Although modern detergents counteract many

of the problems of hard water, some consumers may seek softer, alternative water supplies—many of these supplies cannot provide the assurance of safe water that a closely monitored public water system guarantees.

Concern over soft water. Although a relatively soft water is preferable, in most cases, for consumer use, it does have several disadvantages. Excessively soft water can cause corrosion in pipes. This corrosion can shorten the service life of pipes and household appliances and can result in toxic materials, such as lead and cadmium, being dissolved in drinking water. The problems of corrosion and corrosion control are discussed in Module 9, Stabilization. In addition, if the ion-exchange process is used to soften water, the sodium concentration in the treated water will increase significantly. This is an important health consideration, particularly for people who are on a restricted sodium diet clue to heart disease or hypertension.

The decision to soften a water supply depends completely on the community. Drinking water regulations do not generally require softening—what may be "hard" water in one area may be perfectly acceptable water in another area. Some industries (such as the soft drink and laundry industries) need soft water for their manufacturing processes. Therefore, the decision to soften a water supply should be based on an analysis of the benefits that will be gained against the costs of installing and operating softening equipment. In many cases, it is not economical to soften the entire water supply to meet the special needs of users who could more economically install their own softening equipment.

## Softening Methods

The two methods most widely used for softening public water supplies are:

- Lime-soda ash softening
- Ion-exchange softening.

Each softening process has advantages and limitations. other methods can also be used to soften water, such as electrolysis, distillation, freezing, and reverse osmosis. Although each of these processes can produce softened water, they are complex and expensive processes, used only in unusual circumstances.

**Lime-soda ash softening**. In the lime-soda ash process, lime and soda ash are added to the water and react with various salts of calcium and magnesium to form two insoluble precipitates: calcium carbonate and magnesium hydroxide. These precipitates are then removed by conventional sedimentation and filtration processes. The lime-soda ash process is used in most larger treatment plants. With some modifications, the process can also remove turbidity and color; therefore it should be used when softening surface water. Caustic soda (sodium hydroxide—NaOH) can be used in place of lime and soda ash to remove carbonate and noncarbonate hardness. The major advantage of caustic soda is that it produces less SLUDGE than lime and soda ash. However. since caustic soda is more expensive and increases the amount of total dissolved solids in the treated water, it is not used as widely as lime and soda ash.

Ion-exchange softening. The ion-exchange method removes hardness ions by exchanging or replacing them with sodium ions, which do not cause hardness. In this softening process, ion-exchange materials, such as POLYSTyRENE RESINS. are placed in an ion-exchange unit through which un softened water is passed. As the name suggests, ion-exchange involves the transfer of one ion for another. In this case, calcium and magnesium ions attach to the resin, which releases sodium ions in exchange. With use, the ion-exchange resins lose their ability to remove hardness-causing ions. The resins are then regenerated by passing brine (a salt-water solution) through the ion-exchange unit. The ion-exchange process is used in household softeners and is favored in smaller municipal water treatment plants using ground-water supplies. When a large percentage of water hardness is in noncarbonate form, the ion-exchange softening process is used because the chemical costs of removing noncarhonates are lower with the ion-exchange process than with the lime-soda ash process. The ion-exchange process cannot be easily adapted to treat surface waters.

## Description of Lime-Soda Ash Softening

In the lime-soda ash softening process, lime [$Ca(OH)_2$] and soda ash [$Na_2CO_3$] are added to hard water to remove tile hardness-causing minerals from the water. When lime and

soda ash are added, the hardness-causing minerals form nearly insoluble precipitates. These precipitates are removed by the conventional

Sulfuric acid is sometimes used to accomplish stabilization. Since this process adds noncarbonate hardness ($CaSO_4$) and the acid is more difficult to handle, carbon dioxide is generally used for stabilizing softened water.

## Operating Problems of the Lime-Soda Ash Softening Process

There are several potential problems commonly associated with the operation of the lime-soda ash softening process:

- Excess calcium carbonate
- Magnesium hydroxide scale
- After-precipitation
- Carryover of sludge solids
- Unstable water
- Interference with other treatment processes.

Unless recognized and eliminated, these conditions can lead to high maintenance costs, failure of filtration processes, loss of distribution capacity, inefficient operation of water heaters and boilers, and ineffective treatment.

## Excess Calcium Carbonate

Extremely small, almost colloidal particles of calcium carbonate can be formed during the coagulation/flocculation process. These particles can pass through the entire softening process to the filters, where they adhere very tightly to the individual grains of filter media (sand, anthracite coal, etc.). The filter grains become coated with calcium carbonate and cement together. In a similar way, calcium carbonate can pass through the

media and plug filter under drains. Clogged under drains will usually cause visibly uneven backwash patterns. If these conditions are not corrected, they can cause problems that are serious enough to require complete replacement of the filter media and under drains.

In additional excess calcium carbonate can also precipitate in fine particles on the walls of pipelines.  the particles build up in thickness, reducing the pipeline's capacity. The restricted pipe capacity makes pumps work harder, which increases operating costs and shortens equipment life. Excess calcium carbonate is also he cause of scale deposits on filter walls and wash troughs. Such deposits are both unsightly and costly to remove. These calcium carbonate deposits result from incomplete coagulation/flocculation of lime, which may be caused by inadequate mixing, short detention times, or interferences from organic contaminants in the water during chemical precipitation.

The best solution to the problem is to eliminate whatever is causing the incomplete coagulation/flocculation by improving the mixing process, increasing detention times, or pre treating the water to remove organic contaminants. If the fine particles of calcium carbonate cannot be eliminated at the source, they must either be removed (dissolved) by re carbonation or controlled by applying a recommended dosage of 0.25 to 1.0 mg/L of sodium hexametaphosphate (a sequestering agent). A dosage of sodium hexametaphosphate in this range holds the calcium carbonate in solution, preventing precipitation onto the filter media.

## Magnesium Hydroxide Scale

When water softened by the lime-soda ashprocess contains magnesium

hardness in excess of 40 mg/L, magnesium hydroxide scale can form inside

boilers and household water heaters operated at 140° F (60° C) or higher. This

scale reduces the efficiency of heat transfer, thereby increasing heating costs. A

magnesium hydroxide scale deposit of 0.04 in. (I mm) thick can increase heating

costs by more than 10 percent. To prevent scale buildup, magnesium hardness should be reduced to 40 mg/L or less by using the excess lime treatment.

After-Precipitation

Immediately after the lime-soda ash process is performed, the pH of the treated water is about 9.4—this is the best pH for removal of calcium carbonate. If the pH is left at this level, calcium carbonate will continue to precipitate out of the treated water even after the water leaves the sedimentation basin or upflow basin. This effect, called AFTER-Precipitation, can cause the same problems that occur when excess calcium carbonate precipitates out of treated water due to inadequate coagulation/flocculation. To prevent after-precipitation, carbon dioxide should be added to the softened, settled water in order to lower the pH to about 8.7, a level that provides chemical stability. This procedure converts any unsettled calcium carbonate to soluble calcium bicarbonate. Encrustation can also be prevented by adding sodium hexametaphosphate.

## Carryover of Sludge Solids

The primary reasons that sludge solids and property formed floc particles are carried over the weirs of a sedimentation basin are (I) improper hydraulic conditions and (2) sudden changes in water quality. Carryover is indicated by the unclear water above the sludge blanket, by sludge solids appearing in the basin effluent, and by short filter runs. The specific causes of carryover are listed in Module 4, Sedimentation.

Carryover of sludge solids is often most severe when a considerable portion of the settleable solids is magnesium-hydroxide floc particles, which are very light. To prevent carryover, either reduce the hydraulic loading on the sedimentation or up flow basin or improve the settling characteristics of the floe. Re circulation of previously formed calcium-carbonate sludge is effective in improving the settling characteristics of the floe.

## Unstable Water

If softened water is not properly stabilized, one of two major operating problems will result:

• Scale deposits

• Corrosiveness.

Re carbonation and other stabilization techniques are used to control these operating problems. These techniques are discussed in Module 9, Stabilization.

## Interference with Other Treatment Processes

Disinfection of water is essential to make water safe for consumption. Although the high pH associated with lime-soda ash softening contributes to disinfection, it also results in the formation of chlorine residual that is primarily in the form of hypochlorite ($OCl^-$) when the water is chlorinated. The hypochlorite residual has considerably less disinfecting power than hypochlorous acid (HOCl), which exists at lower p1-I values. Therefore, higher doses of chlorine or longer contact times may be required to disinfect water softened by the lime-soda ash process.

If the lime-soda ash process is used to treat surface water, the processes for removing taste and odor (particularly adsorption) may be hindered due to the increased solubility of taste-and-odor-causing compounds at elevated pH values. If this problem occurs, it can usually he controlled by removing these compounds during pretreatment.

Many surface-water treatment plants must treat raw water having a high color concentration due to organic compounds. For the most part, color must be removed prior to softening since a pH in the range of 4.0 to5,5 is often required for alum coagulation to effectively remove color.

The formation of TRIHALOMETIIANES (THM5) increases at pH levels used in the softening process. If THM problems exist, pretreatment processes may need to be

improved, or disinfection processes may need to be modified. Regulatory agencies or technically qualified consultants should be contacted prior to adopting a plan to minimize THM formation.

## Operational Control Tests

Both the lime-soda ash process and the ion-exchange process require careful and continuous monitoring to effectively soften water. Monitoring is accomplished by performing a series of control tests.

The lime-soda ash process is a complex process, requiring the following control tests:

• Alkalinity

• Total-hardness

• Carbon dioxide

• pH

. Jar lest

• Langelier Index determination for water in the distribution system

• Monitoring of coupons in the distribution system

## pH

Measurements of the water's pH should be made before, during, and following lime-soda ash softening. a pH value of 8.3 denotes the phenolphthalein end point in the alkalinity test. Waters having a pH greater than 8.3 contain phenolphthalein alkalinity (hydroxide), whereas waters having a pH below 8.3 do not.

During the softening process, pH measurements provide valuable information about the success or completeness of the chemical reactions that are occurring. For example, when

excess-lime treatment is used, the lime dosage is designed to raise the pH of the water to 10.6—the best pH for magnesium hydroxide precipitation. The pH is then reduced by recarbonation to 9.4, which is the best calcium carbonate precipitation.

After soda ash is added and softening reactions have been completed, the final H is adjusted by recarbonation to whatever pH is required to achieve stabilization, usually about 8.7. This process converts any residual calcium carbonate into soluble calcium bicarbonate, thus preventing after-precipitation the precipitation of calcium carbonate after softening processes have been completed) either on the filter (filter encrustation) or in the mains (pipeline

By monitoring pH with an accurate. calibrated pH meter throughout the Iteming process, an operator can determine whether or not softening and carbonation dosages are working successfully. In addition, pH and total alkalinity can be used to determine the concentration of carbon dioxide in the

**The jar test** is a good way to test the performance of lime and soda ash dosages fore they are used in the plant. A dosage correctly calculated from the results

**alkalinity test, total-hardness test, and carbon dioxide test** is not necessarily the best dosage. There are numerous other factors, not easily defined

that can greatly influence the selection of the “best” dosage. By :rforming the jar test, an operator can check the overall performance of the e and soda ash dosages, such as the size and condition of floe particles, the

and speed with which floc particles settle, and the resultant final pH. One very useful result of the jar test is that a softening curve can be developed from test results. The softening curve can help .entity the lime dosage that will soften water to the desired point at the least

## Langeller Index and Coupons in the Distribution System

The stability of softened water should he monitored to assure that the water will neither cause corrosion nor form scale. On a daily basis at the plant, the Langelier Index should be determined and evaluated in connection with coupons placed in the distribution system. The Langelier Index and coupons art described in Module 9, Stabilization.

# FILTRATION

The removal of suspended solids by filtration plays an important role in both the naturally occurring purification of ground water and the artificial purification of water in treatment plants.

Filtration takes place naturally as water moves through porous layers of soil. This natural process, called percolation, removes most suspended material from ground water. (If ground waters are treated to remove hardness, iron, and manganese, filtration at the treatment plant' may be needed to remove the chemical precipitates.) Since surface waters are subject to runoff and other sources of contamination, and do not undergo filtration in nature, filtration must be provided at the water treatment plant. At the plant, suspended material is removed and kept from interfering with later treatment processes (primarily disinfection) and from reaching the consumer. This suspended material (measured as turbidity) includes soil, oxidized metals, and microorganisms. Many of these microorganisms are extremely resistant to chlorination and other disinfection methods; however, they can be effectively removed .with filtration.

To help assure effective turbidity removal, a treatment plant operator must understand the fundamentals of filtration. This module describes the types of filtration and common operating proceed Uses used in water treatment. Typical operating problems are presented with discussions of how they can be avoided or solved. The module concludes with a discussion of operational control tests and important safety practices associated with the filtration process..

**After completing this module you should be able to**

- Discuss the role of filtration in water treatment, particularly concerning Public health protection and drinking water standards. .
- Discuss the advantages and disadvantages of slow-sand, rapid-sand, high- rate, and pressure filtration.

- Describe typical operating problems and indicate how they can be avoided or solved.
- Describe dontroi tests that are used to monitor the filtration process.
- Describe safety precautions to be taken during the filtration process.

## Description of Filtration

The major purpose of filtration is to remove suspended material (measured as turbidity) from water. This suspended material can coagulation/flocculation or coagulation, flocculation, and sedimentation processes; microorganisms precipitation such as calcium carbonate that remain after the lime or lime-soda ash softening of ground or surface waters; and iron and magnesium precipitation from ground-water sources.

The suspended materials are removed when water passes through bed of granular material known as the filter media. The filter media is usually sand or some combination of sand .and anthracite coal garnet, or similar substances manufactured s' specifically for water filtration.

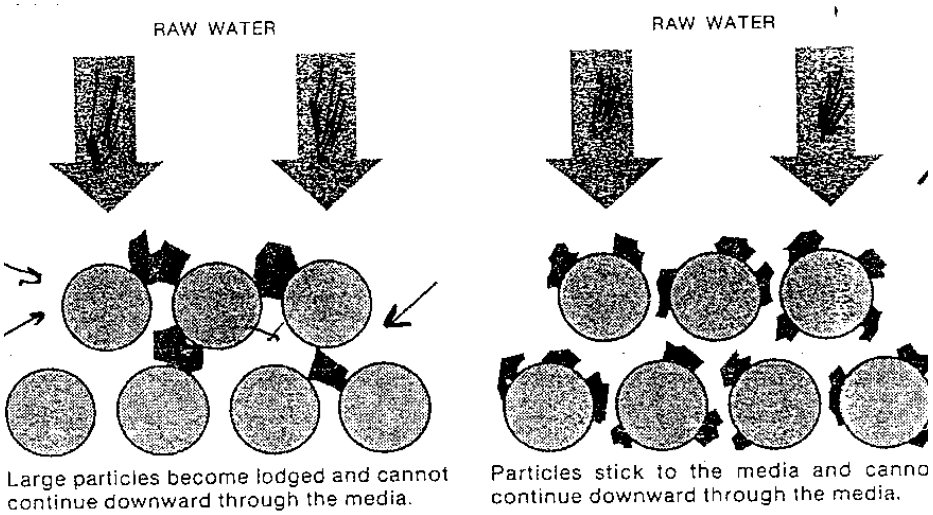
Filtration follows coagulation, flocculation, an sedimentation in the treatment process. /

### Turbidity

Lowering turbidity is important in protecting public health and preventing operational problems in the distribution system .Turbidity interferes with the disinfection process because the suspended particles shield microorganisms from the disinfectant. The particles also combine chemically with the disinfectant and leave less disinfectant to combat the microorganisms. Turbidity cause deposits in the distribution system that create odor and tastes, and bacterial growths, Therefore, lower turbidity results in more effective disinfection and better public health protection.

# Filtration Process

The filter used in the filtration process is commonly thought of as a sieve or micro strainer that traps suspended material between the grains of filter media .However, straining is the least important process in filtration since most suspended particles can easily pass through the spaces between the grains of the filter media.




Fi1tration primarily depends on a combination of complex physical and chemical mechanism the most important being ADSORPTION As water passes through the filter bed, the suspended particles contact and adsorb (stick) onto the surface of the individual filter grains or onto previously deposits materials

The force for attraction between filter media and suspended particles is Van Der Waals force

In fact Some Coagulation and sedimentation occur in the filter bed

This points out the importance of good coagulation before filtration

## Operational Control Tests

Head loss and effluent (filtered-water) turbidity should be monitored and recorded continuously. This provides the operator with the information needed to control the coagulation process, prevent damage to the filter bed, and maintain good-quality filtered water.

A popular method of controlling the direct filtration process is with the pilot filter. Coagulation/ Flocculation, Ely filtering a small stream of the chemically treated water through a miniature filter, the operator can be warned of possible problems before the main stream of water reaches the filters. In a conventional plant, a similar warning can be provided by continuously monitoring the turbidity of the settled water.

A NEPIIEL0METRIC TURHIDIMETER, sensitive to turbidity levels below I NTU, should be used to monitor filtered water. A sample of filtered water from each filter should be tested at least every three to four hours with a laboratory nephelometer to check the continuous monitors and indicate when they need calibration.

The condition of the filter beds must be continuously monitored. In addition to watching for filter cracks, media loss, and other noticeable defects, the filter bed should be periodically sampled. Core samples of the beds can be taken, which are examined to determine the volume of mud balls present. Individual media grains can be examined with a microscope to determine if they are clean.

Determining the mud ball volume is relatively easy and does not require expensive equipment. The only equipment required is a sampler, a 10-mesh sieve, a bucket, and a l000-mL graduated cylinder. After back  washing, the filter is drained until the water is at least (0.3 m) below the sand surface. Five separate samples representative of the filter's surface are taken from the top (150 mm) of sand with the sampler. The samples are placed on the sieve, which is gently raised and lowered in a bucket of water until the sand washes through. Any mud balls will be left on the sieve

The graduated cylinder is filled with water to a defined point. When mud ball are added, the increase in volume in the cylinder represents the mud ball volume. The volume of mudballs should be kept to less than 0.1 percent through proper washing procedures. If a multi-media filter is used, a different sampling procedure is necessary because mud balls will sink through the top coal layer and collect near the sand layer.

Good record keeping for the filtration process can identify problems and indicate proper steps to be taken. the type of filtration records kept will depend on t lie treatment processes being used. All records should included

• Rate of flow, million gallons per day or mega litres per day

• Head loss, feet or meters

• Length of filter runs, hours

• Backwash water rate, gallons per minute

• Volume of wash water used, gallons

• Volume of water filtered

• Length of backwash, minutes

• Length of surface wash, minutes

# ADSORPTION

Water contains varying amounts of dissolved ORGANIC Substances (carbon containing compounds, also called ORGANICS). Many organic materials cause color, tastes, and odors in water and may have serious public health effects. For these reasons, maximum contaminant levels for organic compounds such as PESTICIIWS and Trihalomethane .are included in the USEPA primary drinking water regulations. Removal of these and other organic compounds from drinking water is important in protecting the public health. It is also a major step in making drinking water more pleasing by reducing objectionable tastes, odors And color.

The Adsorption process is used primarily to remove organic materials. Adsorption can also he used to remove the inorganic ions of fluoride and arsenic. The adsorption process may be one of the later steps in treatment, or may be part of the filtration process; or it may be performed as water first enters the treatment plant.

After completing his module you should he able to

• Describe the purpose of adsorption.

• Explain why organic substances should he removed.

• Identify the types of adsorbents used to remove Organics.

• Explain how the adsorption process works.

• List the major operating considerations for using activated carbon.

• Describe common operating problems when using carbon, and recommend appropriate solutions.

Adsorption

## Description at Adsorption

A major goal in water treatment is to produce clear, good-tasting water. Asorption has been used for many years to help achieve this goal by removing organics that cause tastes, odors, and color in drinking water. Recently, it is also becoming important method for protecting public health by removing toxic organics chemicals from drinking water.

### Organics in raw water

Raw water contains varying amounts of dissolved organic materials, with surface water usually having greater concentrations than ground water, since surface water receives runoff and a variety of waste discharges. Some of these organic materials, such as HUMIC SUBSTANCES, occur naturally and are found in most surface waters. A multitude of other organics are introduced when domestic, agricultural, and industrial wastes are discharged into surface water or allowed to contaminate ground water. It is becoming evident that a great deal of round water is being contaminated by organic chemicals, particularly near urban and industrial areas.

In the treatment plant. organics can interfere with treatment processes such as coagulation and flocculation. Organics can also react with chemicals added to water during treatment, primarily chlorine, producing additional undesirable compounds.

### Public Health Concern Over Organics

More than 700 different organic chemicals have been discovered in drinking water throughout the United States. In addition to the taste, color, odor, and treatment problems they cause, long-term exposure to organic compounds may pose a serious threat to public health, including a cancer risk,

Two groups of organic compounds arc of primary concern. The first group is disinfection by-products, primarily trihalomethanes. Trihalomethanes are produced when naturally occurring organic compounds (mostly humic materials) react with the chlorine used in drinking-water treatment. Trihalomethanes, even in very small concentrations, are

suspect CARCINOGENS (cancer causing agents). Other disinfection by-products form when alternative disinfectants, such as ozone or chlorine dioxide, are used. However, not much is known about the public health impact of these compounds.

The second group of organics are man-made and include a variety of chemicals including solvents, cleaning compounds, INSECTICIDES, and Herbicides that enter drinking water sources through waste discharges and runoff. Many of these compounds are known to be toxic to humans. Although federal and state regulations are being revised to control this type of pollution, it will continue to

Type of pollution , it will continue to be a major problem because of accidental discharges, spills, and uncontrollable sources, such as surface runoff from agricultural areas sprayed with insecticides and herbicides,

## Adsorption Methods

Organic chemicals can be partially removed by using chlorine or potassium permanganate to oxidize the compounds, by aeration (which results in oxidation and air-stripping of the compounds), by coagulation/flocculation/ sedimentation, and by filtration. However, these processes cannot remove some organic compounds as efficiently as adsorption.

Adsorption can involve the use of ACTIVATED CARBON or SYNTHETIC RESINS for organics removal, It can also involve the use of ACTIVATED Alumina for the removal of the inorganic elements fluoride and arsenic, However, this module will focus on activated carbon since it is the most commonly used material

### Principle of Adsorption

Adsorption works on the principle of ADHESt0N. In the case of water treatment, organic contaminants are attracted to the adsorbing material (an ADSORBENT such as activated carbon) and adhere (stick) to its surface by a combination of complex physical forces and chemical action,

For adsorption to be effective, the adsorbent must provide an extremely large surface area on which the contaminant chemicals can stick. Also, if the process is to be economical to build and operate, the total surface area needed must be contained in a bed or tank not much larger than that required for a typical rapid-sand or multimedia filter, Porous adsorbing materials help achieve these objectives. Activated carbon is an excellent adsorbent since it has a vast network of pores that vary in size to accept large and small molecules of contaminants. As a result, activated carbon has a very large surface area. For example, I lb (0.5 kg) of activated carbon has a total surface area of about 150 acres (0.6km2), which can trap and hold (adsorb) over ¼ lb (0.25 kg) of carbon tetrachloride.

** These pores are created during the manufacturing process by exposing the carbon to very high heat (about [800° C]) in the presence of steam. This is known as ACTIVATION of the carbon. Activation oxidizes all particles oil the surfaces of the carbon particles, leaving the surfaces free to attract and hold organics. Once the surface of the pores is covered, it loses its ability to adsorb. The spent carbon must then be replaced with fresh carbon. The spent carbon can be REACTIVATED by essentially the same process as activation.

**Activated carbon can be made from a variety of materials such as wood, nutshells, coal, peat, and petroleum residues. Activated carbon used in water treatment is usually made from bituminous or lignite coal. The coal is slowly heated in a furnace without oxygen so that it will not burn. This converts the coal to carbon. The carbon is then activated by exposing it to a steam-air mixture. This activated carbon is crushed and screened to obtain the desired particle size.

There are two common forms of activated carbon used in water treatment—POWDERED Activated carbon (PAC) and Granular ACTIVATED CARBON (GAC).

Table 7-1. A Comparison of GAC and PAC Properties

| Property | Types of Carbon GAC | PAC |
|---|---|---|
| Density, lb/ft3 (g/cm3) | 26-30 (0.42-0.48) | 20-45 (0.32-0.72) |
| Surface area, m2/g | 650-1150 | 500-600 |
| Mean particle diameter, mm | 1.2-1.6 | less than 0.1 |

## Granular Activated Carbon

Granular activated carbon is typically used when carbon is needed continuously to remove organics. GAC is used like filter media. It can be used as a replacement for existing media in a conventional filter, or it can be installed in separate. Closed pressure columns known as Coneacrtor. The decision of which method to use must he based on the types of organic compounds to he removed and the plant layout, For most Situations, 115mg GAC as a partial or complete replacement for the regular filter media is the most economical approach. A minimum of (610 mm) of GA C is suggested to provide effective removal of turbidity and most organic compounds causing tastes and odors, However, deeper depths of GAC are recommended for improved organics removal and to prolong bed life of the carbon (the length of time the carbon is effective in removing organics ). GAC has proven to be as good a filtering media as sand or anthracite coal, and it is available in the same particle—size ranges. ‘[lie depth of carbon possible in a particular plant will depend on how much room the filters have below the wash-water troughs to allow for bed expansion during backwash. In some cases, the troughs may have

## PAC Adsorption

PAC is used primarily to help control those organic compounds responsible for tastes and odors. However, it is also used to remove compounds that will form trihalomethanes if allowed to react with chlorine. In addition, the carbon particies1often aid coagulation by providing Nuclei for proper floc formation, which is an additional benefit particularly where the natural turbidity is tow.

PAC can be applied in the treatment plant at almost any point before filtration. However, the following items should be considered when selecting an application point.

• The contact time between the PAC and the organics is important and depends on the ability of the carbon to remain in suspension; at least 15 mm of contact time should be provided.

• The surfaces of the PAC particles lose their capacity to adsorb if coated with coagulants or other water treatment chemicals.

• PAC will adsorb chlorine,

The above factors indicate that the raw-water intake line is the most advantageous point for PAC addition, if adequate mixing facilities are available. Using a raw-water application point allows a relatively long contact time and avoids the problems that can be caused if PAC is added after prechlorination. The chemical changes that occur when chlorine reacts with organic compounds can produce other compounds, such as trihalomethanes, which are difficult to adsorb. Also, the carbon will adsorb the chlorine and result in increased dosages of both if carbon is added after prechlorination.

If addition to the raw water snot possible, PAC can be added at other points before filtration. but dosages usually-hate to be higher to account for shorter contact time and the interference by other chemicals, such as coagulants and chlorine. If the PAC is added to the effluent from the sedimentation basins or as the water enters the filters, particular care must be taken in filter operation.

The finely divided carbon can pass through the filter and cause black water" complaints 'from customers. An efficient way to apply PAC is to use two or more App1icatinpoints with part bf the carbon being added to the raw water and smaller doses being added later (before filtration) to remove remaining taste- and odor-causing compounds.

The PAC dosage used will depend or the type and concentration of organic compounds that are present. Common dosages range from 2 to 20 mg/ L, but doses can go as high as

100 mg/ L to handle a severe taste-and-odor problem or a spill of an organic chemical such as carbon tetrachloride. The dosage needs lobe established based on the actual water quality and treatment conditions.

## Adsorption

GAC adsorption is used when the removal of organic compounds is necessary a continuous basis and when long contact times are needed for effective removal. The most common procedure is to replace some of the media in a conventional filter with a minimum of 24 in. (610 mm) of GAC. The media is removed using an educator . Once the carbon is placed, the filters should be backwashed to remove entrapped air and carbon fines. The filters are operated the same manner as regular raped -sand filters with a filtration rate of about (1.4mm /s) . This provides an EMPTY

BED CONTACT TIME (EBCT) of 7.5—9 min. Which is sufficient to remove many organic compounds . EBCT is a term used for GAC adsorption beds and is equal to the volume of the empty bed (or space) into which the .GAC will be placed divided by the flow rate through the bed. Therefore, EBCT is calculated much the same way as detention time

The filter bed is backwashed using the same general procedure as for conventional filters. The backwash rate will depend on the grade of GAC (based on particle diameter and density), water temperature, and the amount of bed expansion desired . For proper cleaning of the bed, a 50-percent expansion of the GAC is recommended. Curves are available from GAC manufacturers to help select the proper back wash rate .

Good control of backwashing is very important to achieve adequate cleaning of the bed by preventing the GAC from being washed away. If the GAC has replaced sand, additional care is particularly needed since the particle density of GAC (about 1.4 g/cm1) is much less than that of sand (2.65 g/cm3), If the GAC has replaced anthracite coat, the backwash rate can remain about the same since the particle densities are almost equal.

The two major causes of carbon loss are entrapment within the bed and excessive backwash rates. Prior to backwashing, the filter should be drained to a few inches (millimeters) below the backwash troughs. Surface washers should be turned on for about 2 mm to break up any mat that may have formed on the surface, to break up mudballs within the bed, and to clean floc particles from the carbon.

Some loss of carbon will occur in the backwashing process. It is important to keep track of the bed depth so that the lost carbon can be replaced and so that operating procedures can be changed, if necessary, to prevent serious losses from occurring. Bed depth can be monitored either by taking routine measurements of the distance from the top of the backwash troughs to the top of the carbon bed or by placing a permanent reference mark (such as a stainless-steel plate) in each filter, to indicate where the top of the carbon should be. It is not unusual for approximately (25 mm) of carbon per year to be lost from the filters.

The actual life of tie GAC bed depends on the concentration and Type of organic compounds being removed. For typical taste- and odor-causing compounds, bed life may be as long as three years. However, for trihalomethane compounds such as chloroform, bed life can be as short as one month, Generally speaking, as the influent concentration increases the bed life decreases. In addition, some organic compounds are not as strongly adsorbed as others, depending on their molecular weight and other physical-chemical properties. For example, pesticides, phenols, and taste- and odor-causing compounds are strongly adsorbed, while alcohols and trihalomethanes are weakly adsorbed. The more weakly adsorbed a compound is. The shorter the bed life will be.

A major advantage GAC beds have is that they are not all used up at once; therefore, breakthrough of the contaminants takes place over a long period of time rather than suddenly. This pattern is important since it allows the replacement of the GAC to be phased so all of the filters do not have to be out of service at the same time. The spent GAC is removed with an eductor to a truck for hauling to a disposal or reactivation site. Methods to help determine when to replace the GAC are discussed under Operational Control Tests and Record Keeping.

GAC can also be placed in a closed container much like a pressure filter. These containers are known as contactors or absorbers. They are used primarily to remove contaminants such as trichloroethylene (TCF) from ground water. In these cases the water is pumped from the wells and through the contactors.

An advantage of using contactors is that they can be manufactured to the size necessary to provide the desired EBCT. As a result, contactors might also be very cot effective in surface-water systems that have high concentrations of organic compounds particularly those that are weakly adsorbed. Under these conditions, a longer EBCT is needed than the 2 ft (0.6 m) of GAC in a gravity filter can provide while still maintaining a reasonable bed life

The decision to use contactors and the contactor design should be based on pilot tests at the treatment plant where they will be installed. If used in a conventional surface-water treatment plant, the contactors should be placed after the filtration process. This will allow the suspended matter to be removed and (he GAC 10 be used primarily for adsorption. The life of the GAC will be extended because it will not become coated with floe particles and will not have to be backwashcd as often.

Because not many contactors are in full-time use, there is little operational experience available. Usually, two or more contactors are operated in parallel down flow operation, like gravity filters. However, the start-up of the units is staggered so that (lie beds will not all he exhausted at the same time. The beds

| *Bed Depth* ft | m | *EBCT* min | *Average Influent Chloroform Concentration* mg/L | *Time to Bed Exhaustion** Weeks |
|---|---|---|---|---|
| 2.5 | 0.8 | 6.2 | 67 | 3.4 |
| 5.0 | 1.5 | 12 | 67 | 7.0 |
| 7.5 | 2.3 | 19 | 67 | 10.9 |
| 10.0 | 3.0 | 25 | 67 | 14.0 |

*Bed is exhausted when effluent concentration is equal to influent concentration.

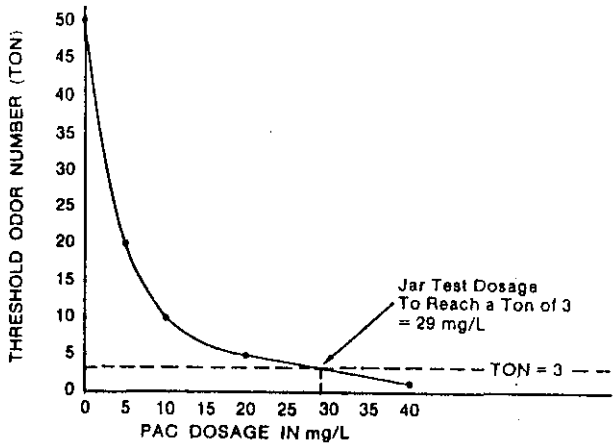
## Control Tests

Operational control tests for adsorption range from jar tests to sophisticated water quality analyses using a gas chromatograph. Although analyses for specific organic compounds or general organic parameters such as TOTAL ORGANIC Carbon (TOC) are highly recommended . many treatment plants do not have the a analytical equipment or laboratory personnel to perform these tests. This section will discuss operational tests that can be performed by the operator.

### Powdered Activated Carbon

The major operational control tests for a PAC system are (I) determination of the proper dosage and (2) monitoring of the treated water to determine if organic compounds are being removed.

Dosage can be determined with a modification of the jar test used for coagulants. The stirring apparatus as well as all critical glassware should be cleaned with a non scented detergent and rinsed thoroughly with odor-free water. One-liter samples of the raw water are then dosed with varying amounts of a well-shaken stock IAC suspension, such as 5, 10, 20,and 40mg/ L. The stock solution is prepared by adding I g of PAC to I L of odor-free water. Each milliliter of this solution when added to a l-L sample of raw water is equal to a dosage of I mg/L. The four dosed samples and a fifth sample to which no PAC is added are stirred for a time period that approximates the contact time the PAC will have with the water ip the plant. At the end of that time, each sample is filtered through glass wool or filter paper to remove the PAC. The first 200 ml of sample through the filter is discarded and the remainder subjected to the threshold odor test to arrive at a threshold odor number (TON) for each sample. The results can then be plotted to obtain an optimum PAC dosage. In the example, the dosage needed to reduce the TON to 3




would be 29 mg/ L. Experience has indicated that plant-scale application is more efficient than that obtained by the jar test. 1-fowever, the plant should begin with the dosage indicated by the jar-test result and then gradually reduce it. The threshold odor test on the raw and treated water should be conducted daily while tile PAC is being fed, to determine if changes in the dosage are required.

If PAC is used to remove compounds other than those causing tastes and odors, different procedures arc needed to determine proper dosage. For example, if the goal is to remove trihalomethane Precursor COMPOUNDS ( humic substances that react with chlorine to produce THMs), then pilot testing should be done to determine the removal of those specific compounds.

Good record keeping can help prevent taste- and odor-causing compounds from reaching the consumer and prevent the wasting of. PAC. The records should include the dates on which PAC was being added, the calculated dosage (from the jar test), the actual plant dosage that was effective, and the TON values for the raw and treated water. The amount of PAC fed should be calculated and recorded each day to help determine the time to order more chemical. It is also helpful to record the type of odor or taste in the water—for example, musty. septic, or rotten-egg odors and sweet or bitter tastes. This might give a clue as to the source of the problems so that preventative action can be taken at the source. All of these records will give the operator a history of the problems that have

occurred each year and allow planning so proper addition of PAC can be accomplished when it is needed.

## GAC Used In Conventional Gravity Filters

Since use of GAC in a conventional gravity filter is a combination of filtration and adsorption, most of the control tests are those used for filtration. Head loss and turbidity of the filtered water should be monitored and recorded continuously to ensure proper backwashing frequency and good quality water. The condition of the lifter bed should be checked frequently, particularly during and after backwashing, to determine if there are problems with cracks, shrinkage, channeling, or mudballs.

In addition, certain additional tests should be conducted to monitor the adsorption process. The distance between the top of the wash-water trough (or some other predetermined reference point) and the top of the carbon should be measured and recorded at least every three months to determine the rate of carbon loss. Samples of backwash water can also be tested to see if excessive carbon is being lost. Excessive loss may show a need for changes to the backwashing procedures. Core samples of the carbon bed should be taken at the time of installation and at least every six months to determine the amount of bed life remaining. The sample should represent the carbon from top to bottom of the bed. GAC manufacturers can provide the necessary information on how to sample the bed, what tests can be used, and what the tests mean.

If the filter is removing taste- and odor-causing compounds, the threshold odor test should be conducted routinely on the raw and finished water as a check on the effectiveness of the filter. lithe filter is intended to remove Other types of organic compounds, analyses should be conducted for these compounds.

Since bacteria can thrive in GAC filters, standard-plate-count analyses should done on the filtered water and the water alter final chlorination each day. This will help determine the final chlorination dose and will indicate whether chlorination of the backwash water is needed. Tl1e plate count should be kept well below 500 organisms/ml

In addition to the records recommended for the filtration process . Records should be kept on the amount of carbon lost, the bed life of the GAC, and the results of the threshold odor test and, other analyses performed.

## GAC Contactors

Since GAC contactors are used for organic compounds other than those causing tastes and odors, more sophisticated control tests are usually needed. Generally, a specific compound or a group of compounds (such as trihalomethanes) is remove & therefore, the effectiveness of the Contactors must be judged by testing for those compounds. This requires special laboratory equipment and trained personnel. In addition to these chemical analyses, monitoring of other factors is required. Head loss through the Contactors should be monitored and recorded continuously, so that backwashing is performed at the proper time. Core samples of the carbon bed should be taken at least every three months so that approximate remaining bed life can be determined. Turbidity of the contactors' effluents should be monitored and recorded continuously to determine if carbon fines or other suspended matter are passing through the GAC bed. Standard plate count analyses of the water from the conventional filters, the contactors, and after final chlorination should be conducted daily to help determine what chlorine dosages are needed in the plant to control bacterial growth.

In addition to the above, records should be kept of the amount of water treated, the bed life of the GAC. the time when a Contactor is backwashed, and the amount of backwash water used. When fresh GAC is placed in a contactor the amount added should be recorded so losses can be calculated.

## Safety and Adsorption

Since PAC and GAC are fine, dusty materials, care must betake in handling and storing them. If either PAC or GAC are being used in bags, the bags should be stored in a clean, dry place on pallets so air can circulate underneath. The bags should be stacked in single or double rows with access aisles around every slack to allow easy handling and fire inspections. Carbon bags should never be stored in stacks higher than (2 m). Although

activated carbon, even as a dust, is not considered explosive, it will burn. Like charcoal used in barbecues, it burns without producing smoke or flame and glows with the release of intense heat. Such fires are difficult to detect and when found are hard to handle. This is a major reason for using slurry storage if large quantities of carbon are being used.

The storage area should be fireproof, with self-closing fire doors separating the carbon storage area from other areas. Storage bins for dry bulk carbon should be fireproof and equipped for fire control.

Burning carbon should never be doused with a large stream of water since the fire will be spread quickly as the burning carbon particles fly in all directions. A fine mist or spray from a hose, or chemical foam from a fire extinguisher are effective in controlling carbon fires. Also, smoking should be prohibited at all times during the handling and unloading of carbon and in the storage and feeding areas.

Activated carbon should never be stored along with gasoline, mineral oils, or vegetable oils. Such materials, when mixed with carbon, will slowly oxidize until ignition temperature ( [316—427°C]) is reached, Never mix or store carbon with chlorine compounds or potassium permanganate .since spontaneous combustion can occur, Carbon should not be stored where overhead electrical equipment could start a fire,

Because activated carbon is an electrical conductor, dust must be minimized. The heat from a motor may ignite the accumulated dust, which could short- circuit the motor and cause a fire. Therefore, all tanks receiving dry carbon should be vented and provided with dust-control equipment, such as bag-type dust collectors, Even slurry storage tanks should be so equipped. If a PAC dry-feed system is used, the hoppers and feeders should be enclosed in a separate room so dust will not spread throughout the plant. As an additional precaution, the electrical equipment should be cleaned frequently. Explosion-proof electrical wiring and apparatus should be used for any equipment operated near carbon storage areas, especially while feeding is being performed.

Since oxygen is removed from air in the presence of wet activated carbon, slurry tanks or other enclosed spaces containing carbon may have seriously reduced oxygen levels, Therefore, care should be taken when entering such spaces.

# STABILIZATION

Many waters either cause corrosion or deposit. scale on pipelines and plumbing fixtures, Such waters arc said to be unstable. The treatment process used to reduce or eliminate the problem of corrosion and scaling is stabilization. Stabilization is normally one of the lat processes in the treatment plant. water Stabilization requires a basic understanding of the chemistry of the treatment processes used in he plant and an ability to monitor and control treated water chemistry within relatively narrow limits. Monitoring and control at the plant and in the distribution system are essential to the success of Stabilization.

After completing this module you should he able to

• Describe the purpose of stabilization.

. Identify the primary problems caused by unstable waters -

• Identify the different operating methods practices in controlling unstable water.

• Identify typical operating problems and solutions..

• Describe and perform the common operational control tests recommended for monitoring stabilization processes.

• Explain the basic safety considerations elated to the stabilization processes

## Description of Stabilization

### Purpose of Stabilization

The purpose of stabilization is to control both corrosive and scale-forming tendencies of drinking water before the water enters the distribution system. Distributing unstable water can cause problems related to public health, aesthetics, and economics.

Public health problems. Corrosive water can dissolve toxic metals into the drinking water from distribution and household plumbing systems. Lead and cadmium are the metals most likely to be a problem, since they are commonly present in household plumbing. Lead service lines are common in older water systems and lead solder is used to join copper pipes in household plumbing. Cadmium is a constituent of galvanized piping. Corrosive water in contact with either lead or cadmium can result in concentrations of metals that pose a threat to public health.

Corrosion of cast-iron mains in the distribution system can cause the formation of iron deposits, called Tubercules. in the mains. These deposits protect bacteria and other microorganisms from residual chlorine, allowing large populations of microorganisms to thrive. Pressure or velocity changes of the water in the main can cause the microorganisms to be released into the water, creating a potential for disease outbreaks. Sonic types of bacteria can also accelerate the corrosion process.

Aesthetic problems. Color, taste, and odor problems can all result from corrosive water attacking metal pipes in the distribution and household plumbing systems. RED WATER problems can occur when iron is dissolved from cast-iron mains by corrosive water. The dissolved iron stains plumbing fixtures and laundry and makes water unappealing. The dissolved iron also acts as a food for a group of microorganisms. called IRON Bacteria, which can cause serious taste and odor problems. Corrosion of copper pipe can result in water with a metallic taste and can cause blue-green stains on plumbing fixtures and laundry.

Economic problems. Unstable water results in significant costs to water systems and customers each year. Aggressive water can significantly reduce the life of valves and unprotected metal or asbestos—cement pipe in the utility distribution system, as well as affecting the service life and performance of consumer's plumbing systems and hot—water heaters. Buildup of corrosion products (tuberculation) or uncontrolled scale deposits can seriously reduce pipeline capacity and increase resistance to flow. This, in turn, reduces distribution-system efficiency and substantially increases pumping costs. If scale deposits or tuberculation go unchecked pipes may become completely plugged,

requiring expensive repair or replacement. Sealing can also cause operation of hot-water heaters and boilers to become more expensive due to reduced volume and heating capacity

In many systems. monitoring and control of water stability has been neglected because personnel are unaware of the extensive damage that can result from unstable water. Inmost cases, the costs of monitoring and stabilizing the water are outweighed by the savings associated with more efficient system operation and reduced maintenance.

The US Environmental Protection Agency has established a primary drinking water regulation that requires monitoring to determine water stability, If the water is found to be corrosive, the regulations suggest that a control plan be developed. In addition, the Secondary Regulations of the Safe Drinking Water Act indicate that drinking water should be none corrosive.

## Chemistry of Water Stability

The stability of treated water depends on several chemical characteristics. To understand how these characteristics can be adjusted to control stability, it is necessary to understand the basic principles of corrosion and scale formation. Corrosion. Corrosion can be broadly defined as the wearing away or deterioration of a material due to chemical reactions with its environment. The most familiar example of this is the formation of rust—oxidized iron—when an iron or steel surface is exposed to moisture. Corrosion is usually distinguished from EROSION, the wearing away of material due to physical causes such as abrasion. Water that promotes corrosion is known as CORROSIVE or AGGRESSIVE water.

In water treatment operations, corrosion can occur to some extent with almost any metal that is exposed to water. Concrete and asbestos—cement can also be affected by aggressive water, which will dissolve the cement. However, this process involves leaching rather than the electrochemical corrosion reactions that take place with metal pipe. Whether corrosion of a material will be so extensive as to cause problems depends on several interrelated factors, including the material involved, the chemical and

biological characteristics of the water, and the electrical characteristics of the material and its environment. The interrelation of these factors, as well as the process of cot rosion itself, is quite complex. As a result, it is difficult to make general statements about what combinations of water and equipment will or will not have corrosion problems.

**Chemistry of corrosion**

The chemical reactions that occur in the corrosion of metals are similar to those that occur in an automobile battery. In fact, corrosion generates an electric current, which flows through the metal being

corroded. The chemical and electrical reactions that occur during CONCENTRATION CEll CORROSION of iron pipes are illustrated in Figure

In Figure, minor impurities and variations in the metal (found in all metal pipes) have caused one spot on the pipe to act as an electrical ANODE in relation to another spot that is acting as an electrical CATHODE. At the anode, atoms of iron ($Fe_2$) are breaking away from the pipe and going into solution in the water. As each atom breaks away, it ionizes by losing two electrons, which travel through the pipe to the cathode.

In Figure chemical reactions within the water balance the electrical and chemical reactions at the anode and cathode. Many of the water molecules ($H_2O$) have dissociated into H+ ions and OH- radicals—this is a normal condition, even with totally pure water. The Fe2released at the anode combines with two $OH^-$ radicals from dissociated water molecules to form $Fe(OH)^2$ ferrous hydroxide. Similarly, two H+ ions from the dissociation of the water molecule near the cathode pick tip the two electrons originally lost by the iron atom, then bond together as H2. hydrogen gas.

The formation of $Fe(OH)_2$ leaves an excess of $H^+$ near the anode, and the formation of the H2 leaves an excess of $OH^-$ near the cathode. This change in (he normal distribution of H' and OH- accelerates the rate of corrosion and causes increased pitting in the anode area (the "concentration cell"), as shown in Figure If the water contains dissolved oxygen (02), which most surface water does, then $Fe(OH)_3$, ferric hydroxide. will form (Figure ).

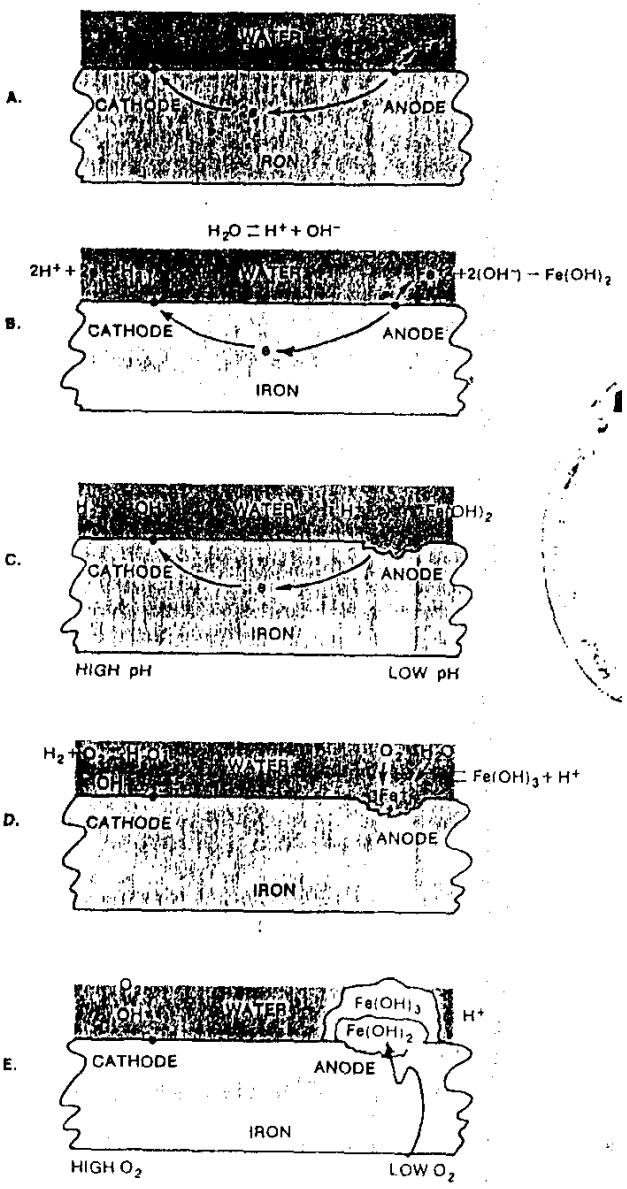
Ferric hydroxide is common iron rust. The rust precipitates, forming deposits called tubercules . The existence of tubercules further concentrates the corrosion, increasing both pitting at the anode and growth of the tubercule. Tubercules can grow into large nodules . Significantly reducing the carrying capacity of the pipe. During rapid pressure or velocity changes, some of the $Fe(OH)_3$ can be carried away, causing “red water.”

**Factors affecting corrosion.**

The rate of corrosion depends on many site pacific conditions, such as the characteristics of the water and pipe material. therefore, there arc no established guidelines that determine the rate at which a e will be corroded.

Chemical reactions play a critical role in determining the rate of corrosion at both the cathode and anode. Any factor that influences these reactions will also influence the corrosion rate. Factors that affect the stability of water include:

- Dissolved oxygen
- Total dissolved solids (TDS)
- pH
- Alkalinity
- Temperature
- Electrical insulation and stay electric currents
- Bacteria.




Dissolved oxygen (DO) is a key part of the corrosion process; therefore, as the concentration of DO increase the corrosion rate will also increase. The TDS concentration is important because electrical how is necessary for the corrosion of metal pipe to occur. pure water is a poor conductor of electricity since it Contains few ions to carry electric cm rent. As the UPS concentration increases, water becomes a better CL) ii cl tie to tic re by increasing the corrosion rate. Alkalinity and pH also affect the rate of

chemical reactions. Generally, as p11 and alkalinity increase, the Corrosion rate decreases.

Since chemical reactions occur faster at higher temperatures, an increase in water temperature generally increases the corrosion rate. Flow velocity can increase or decrease the corrosion rate depending on the nature of the water. If the water is corrosive, higher Flow velocities cause turbulent conditions that bring dissolved oxygen to the corroding surface more rapidly, which increases the corrosion rate. However. If chemicals are being added to stabilize the water. The higher velocities “ill decrease the corrosion rate by allowing the chemicals (such as $CaCO_3$) to deposit on the pipe walls more quickly.

Metals that easily give up electrons will corrode easily. Where dissimilar metals are electrically connected and immersed in a common flow of water, the metal highest in I he galvanic series will immediately become the anode, the other her metal will become t lie cathode, and corrosion will occur. This is termed GALVANIC CORROSION. The rate of galvanic corrosion will depend largely on how widely separated the metals are in the galvanic series —widely separated metals will exhibit extremely rapid corrosion of the anode metal (the metal highest in the series); the cathode metal will be protected from corrosion. A common example

**Table 9-2. Galvanic Series For Metals Used In Water Systems**

| Corroded End (Anode) | MOST ACTIVE |
|---|---|
| Magnesium | + |
| Magnesium alloys | |
| Zinc | |
| Aluminum | |
| Cadmium | |
| Mild steel | |
| Wrought (black) iron | |
| Cast iron | |
| Lead-tin solders | |
| Lead | |
| Tin | |
| Brass | |
| Copper | |
| Stainless steel | − |
| **Protected End (Cathode)** | **LEAST ACTIVE** |

Corrosion Potential

of galvanic Corrosion occurs when a brass corporation cock is tapped into a cast-iron main and attached to a copper service line. The copper will be protected at the expense of the brass and east iron.

The electric current generated by corrosion is very slight, and insulation that interrupts the electrical current between dissimilar metals will prevent galvanic corrosion. However, when a strong current is passed through any corrodible metal, usually due to stray currents from improperly grounded household electrical systems, rapid corrosion will result.

Certain types of bacteria can accelerate the corrosion process because they produce carbon dioxide ($CO_2$) and hydrogen sulfide ($H_2S$) during their life cycles. Which can increase the corrosion rate. They can also produce slime, which will entrap precipitating iron compounds and increase the amount of tuberculation and red—water problems. Two groups of bacteria ca use the most problems. Iron bacteria. Can forming considerable amounts of slime on pipe walls particularly if the water has enough dissolved iron present (from the source water or from corrosion of the pipe material to allow them to thrive. Beneath this layer, $CO_2$ production by the bacteria can significantly lower the pH, which will speed tip the corrosion rate. The periodic sloughing of these slime accumulations can cause other major problem such as tastes and odors. The slimes can also prevent the effective deposition of a protective $CaCO_3$ lining by enmeshing it within the slime layer. As this layer sloughs away it carries away the $CaCO_3$ and leaves the pipe surface bare. Sulfate—reducing bacteria. Can accelerate corrosion when sulfate ($SO_4$) is present in the water. They reduce the SO tinder anaerobic conditions (which occur tinder the slime layer where the oxygen is depleted by other bacteria) to form H2S and iron sulfide ($Fe_2S_2$) causing obnoxious odors and a black-colored water. Carbon dioxide production can also lower the pH.

All of these factors interact with each other and the pipe material. This combination of factors and their influence on the corrosion reactions will help

determine how quickly the corrosion proceeds. As the factors change, the corrosion rate will also change. the only factors over which the operator has significant control are pH. alkalinity, and the bacteriological content of the water. The techniques used to control these parameters are discussed later in this module.’

Types of corrosion. Corrosion in water systems can be divided into two broad classes—localized and uniform. LOCALIZED Corrosion, the most common type in water systems, attacks metal surfaces unevenly. Localized corrosion is usually a more serious problem than uniform corrosion since it leads to a more multi fail tire of I he pipe. Two common types of corrosion that produce pitting arc galvanic corrosion and concentration—cell corrosion, discussed in the preceding paragraphs. UNIFORM CORROSION takes place at an equal rate over (he entire surface. it usually occurs where waters having very low pH and low alkalinity act on un protected surfaces.

Scale formation. Scale-forming water can protect pipes iron corrosion by depositing a layer of scale that separates the corrodible pipe material from the water. However, uncontrolled scale deposits can significantly reduce the carrying capacity of the distribution system .

**Chemistry of scale formation :-**

Scale is formed when the divalent metallic cations ' associated with hardness, primarily magnesium and calcium, combine with other minerals dissolved in the water and precipitate to coat pipe walls . The most common form of scale is calcium carbonate ($CaCO_3$). Other scale-forming compounds include magnesium carbonate ($MgCO_3$), calcium sulfate ($CaSO_4$), and magnesium chloride ($MgCI_2$).

**Factor effecting scale formation :-.**

Water can hold only so much of any given chemical in solution—if more of the chemical is added, it will precipitate instead of dissolve. The point at which no more of the chemical can he dissolved is called the SATURATION POINT. The saturation point varies with other chemical characlcristies of the water, including ph-I, temperature, and total dissolved solids (TDS),

The saturation point of calcium carbonate depends primarily on the ph of the water. For example. If a water with a certain temperature and TDS concentration can maintain 500 mg/ L of CaCO3 in solution at pH 7, then the same water will hold only 14mg! L of

CaCO3 in solution if the pH is raised to 9.4. Temperature also affects the saturation point, although not as dramatically as pH. The solubility of CaCO3 in water decreases as temperature increases. The increased temperature in hot-water heaters and boilers causes major problems with scale, which significantly reduces their energy efficiency. Since the presence of other minerals in the water affects the solubility of CaCO3 the TDS concentration must he known in order to determine the CaCO3 saturation point. As the TDS concentration increases, the solubility of CaCO3 increases.

## Stabilization Methods

The basic methods used for stabilizing water or protecting the distribution system against unstable water is:

• Adjustment of ph and alkalinity

• Use of protective coatings

• Use of corrosion inhibitors and sequestering agents.

Selection of the method or methods to be used in a given application depends on the chemical characteristics of the raw water and on the effects of other treatment processes used.

### Adjustment of pH and alkalinity.

An increase in pH and alkalinity levels can prevent corrosion; a decrease can prevent scale formation. Li me is generally used to increase both pH and alkalinity since it is less expensive than other chemicals having the same effect. Soda ash (sodium carbonate) may be added along with the lime to further increase the alkalinity. Sodium carbonate is often used instead of sodium carbonate since it will also increase alkalinity without as much increase in pH. The increased alkalinity buffers the water pH changes in the distribution system. This has proven particularly effective in controlling corrosion of lead and copper service pipes. Instead of lime, caustic soda with soda ash or sodium bicarbonate can also be used to increase pH and alkalinity.

Lime-softened water can cause severe scale problems ii it is nut stabilized. Stabilization after softening is accomplished by adding carbon dioxide (re carbonation) or sulfuric acid. Both chemicals lower pH so calcium carbonate will not precipitate in the distribution system.

Use of protective coating. Since corrosion attacks the surface of the pipe, a protective Coating on (the pipe surface can inhibit corrosion. A lining of cement or asphalt material is commonly applied to cast—iron and other distribution during the manufacturing process. Although these coatings form an effective barrier against corrosion, aggressive water can attack the coatings or reach the pipe where gaps in the coating occur. As a result, many systems apply an additional protective coating by controlling the chemistry of the water through the pipes.

**protective coatings**

A common protective-coating technique is to adjust the pH of the water to a level just above the saturation point of calcium carbonate. When this level is calcium carbonate will precipitate and form a protective layer on the pipe walls. This process must be closely controlled—a pH that is too low may result in corrosion, a rid a p H that is too high may result in excessive precipitation, causing clogging of service pipes and increased flow restriction in the distribution system. lime, soda ash (sodium carbonate), sodium bicarbonate, or sodium hydroxide can be used to raise the pH level. Lime is often used since it also adds needed calcium (hard ness) and alkalinity.

**corrosion inhibitors and sequestering agents.**

If waters do not contain enough calcium or alkalinity to make the formation of calcium carbonate coatings economical (water obtained primarily from snowmelt is an example, since it can have alkalinity and calcium concentrations as low as 2 mg/L as caCO3), then other chemical compounds can be used to form protective coatings. The most common compounds are polyphosphates and silicates. The chemical reactions by which these compounds combine with corrosion products to form a protective layer are not completely understood; however, the chemicals have proven successful in many systems.

Some polyphosphates can also be used as Sequestering AGENTS to prevent scale formation, these compounds sequester, or chemically tie up, the scale— forming ions (primarily calcium and magnesium) so they cannot react to form scale. Since these compounds remain in solution they are eventually ingested by the consumers. Therefore, any sequestering agent selected must be suitable for use in drinking water.

Polyphosphates also sequester iron, whether it is dissolved in water from the source or from corrosion of the system. This prevents the precipitation of the iron compounds so red water will not result. 1-lowever, this effect does not prevent corrosion —it inc rely prevents the corrosion by—products (iron in this case) from causing additional problems.

Whatever method or methods are used for stabilization, the facilities required for the process consist primarily of selected chemicals and the equipment required to store, handle, and feed these chemicals. A wide range of chemicals is available for stabilization. Some of these arc primarily intended for industrial or other non potable applications. Therefore, all chemicals used to stabilize drinking water must be suitable for such use.

**Table 9-3. Chemicals Commonly Used For Stabilization of Potable Water**

| *Treatment Method* | *Chemical Name* | *Chemical Formula* |
|---|---|---|
| Increase pH and alkalinity | Lime<br>—Unslaked lime (quicklime)<br>—Slaked lime (hydrated lime)<br>—Sodium bicarbonate<br>—Sodium carbonate (soda ash)<br>—Sodium hydroxide (caustic soda) | <br>$CaO$<br>$Ca(OH)_2$<br>$NaHCO_3$<br>$Na_2CO_3$<br>$NaOH$ |
| Decrease pH and alkalinity | Carbon dioxide<br>Sulfuric acid | $CO_2$<br>$H_2SO_4$ |
| Formation of protective coatings | Sodium silicate (water glass)<br>Sodium hexametaphosphate (sodium polyphosphate, glassy)<br>Sodium zinc phosphate<br>Zinc orthophosphate | $Na_2O(SiO_2)_n$*<br>$(NaPO_3)_n \cdot Na_2O$†<br>$(MPO_3)_n \cdot M_2O$‡<br>$Zn_3(PO_4)_2$ |
| Sequestering agents | Sodium hexametaphosphate (sodium polyphosphate, glassy)<br>Tetrasodium pyrophosphate | $(NaPO_3)_n \cdot Na_2O$†<br>$Na_4P_2O_7 \cdot 10H_2O$ |

*Typically n = 3.
†Typically n = 14.
‡M = Na and/or 1/2 Zn, typically n = 5.

# DISINFECTION

One of the operator's most important roles in water treatment is to deliver water that is free from disease-causing (Pathogen) organisms. Disinfection is the treatment process used to destroy these organisms, usually through the addition of chlorine to the water. Diseases caused by pathogenic organisms in water are called WATER BORNE DISEASES; the more common ones are summarized in Table. As the table indicates, the consequences of waterborne disease can range from mild illness to death. Clearly, the importance of disinfection cannot be overemphasized. (Disinfection—the destruction of disease-causing microorganisms—should not be confused with STERILIZATION, which is the destruction of all living microorganisms.)

After completing this module you should be able to

• Define the three general types of disinfection.

• Discuss the different chemicals used for disinfection along with their advantages and disadvantages.

• Understand the general chlorination chemical reactions.

• Describe and plot a typical chlorination breakpoint" curve.

• Identify and discuss the five factors important to successful chlorination.

• Describe the three chemicals used for chlorination.

• Discuss typical operating problems with chlorination and possible solutions.

• Describe the operational control tests and record keeping suggested for effective chlorination.

• Describe the safety practices that must be followed when using each chlorine chemical.

## Destroying Pathogens in Water

CHLORINATION, usually performed as the final treatment process (Figure 10-I), is the most common means of disinfecting drinking water. Other processes, natural and artificial, aid in the destruction or removal of pathogens

Most pathogens are accustomed to living in the temperatures and conditions found in the bodies of humans and animals; they do not survive well outside the body. Nonetheless, significant numbers can survive in potable water. Some pathogens. particularly certain viruses and those organisms that form cysts, can survive for surprisingly long periods even under the most adverse conditions. Because such organisms also tend to be resistant to chlorine doses normally used in water treatment, chlorination alone cannot always ensure safe drinking water.

Various Treatment Processes

| *Unit Process* | *Percent Reduction* |
|---|---|
| Storage* | Significant amounts |
| Sedimentation* | 0–99 |
| Coagulation* | Significant amounts |
| Filtration* | 0–99 |
| Chlorination | 99 |

*These methods do not, in themselves, provide adequate pathogen reduction. However, their use prior to disinfection may significantly lower the costs associated with disinfection.

Storing water for extended periods in open tanks or reservoirs prior to treatment can accomplish sonic destruction of pathogens through sedimentation and natural die-off of the organisms. Significant pathogen removal also occurs during the conventional treatment processes of coagulation, flocculation, sedimentation, and filtration.

All drinking water should receive some type of disinfection to help ensure thu pathogens are not present. Because surface waters are more susceptible Iii contamination than ground waters, they should always receive treatment (such as coagulation, flocculation, sedimentation • and filtration) before disinfect in and delivery to consumers.

## Methods of Disinfection

Although chlorination is the most common technique for disinfection, o lie methods are available and may be useful in some situations; this section briefly summarizes the methods available.

Three general types of disinfection arc:-

- Heating treatment
- Radiation treatment
- Chemical treatment.

**Heat treatment**. Heat was probably the first method used for disinfection it is still a good emergency procedure for small quantities of water. To disinfection with heat, bring water to a boil and continue to boil it for 5 to 20 mm. The boiling time varies with altitudes—at higher altitudes more time will be needed because boiling temperature is lower. Boiling can to disinfect drinking water at a campsite or to disinfect household drinking water supplies when a local health agency has issued a 'boil order.'' Such an order could be issued whenever the bacteriological quality of the water does not meet drinking water standards. Boiling is not suited for large-scale use. primarily because of the high cost of energy required.

**Radiation treatment**. Sunlight is a natural means of disinfection—the ultraviolet portion of t lie sun's light destroys microorganisms. Ultraviolet radiation can also be produced from specially designed lamps: these have had limited use in water disinfection. The use of ultraviolet radiation to destroy pathogens, called UV DISINFECTION, is costly. Since UV light is readily absorbed and scattered by the turbidity-causing impurities in water, the light's killing power is greatly diminished in water that does not pass close to the lamp. These disadvantages and the lack of measurable residual have discouraged widespread use of UV disinfection.

**Chemical treatment**. The chemicals used as disinfectants are:

• Bromine

• Iodine

• Ozone

• Chlorine and chlorine compounds.

**Bromin:** Bromine is a dark reddish-brown liquid. It vaporizes at room temperature and has a penetrating, suffocating odor. The vapor is extremely irritating to the eyes, nose, and throat, and it is very corrosive to most metals. When spilled or splashed onto the skin, bromine causes painful burns that are slow to heal.

The residual formed when bromine is added to water is as effective a disinfectant as chlorine, but not as stable. Consequently, depending on the constituents in the water being treated, it may be necessary to add bromine at two or three times the concentration required for chlorine. Because of the added cost factor and the handling hazards, liquid bromine is not used to disinfect municipal water supplies; it is used in the safer, but more costly, solid “stick form” (ORGANOBROMINE COMPOUND) to disinfect swimming pools.

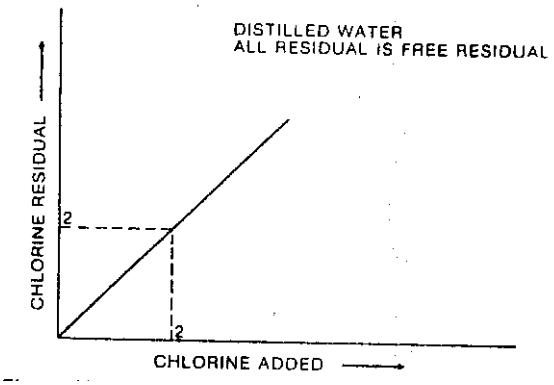
**Iodine:** Iodine is a lustrous, blue-black solid, about five times the density of water, with a peculiar chlorine-like odor. The solid can quickly change to a gas, releasing a characteristic violet vapor. Iodine has been used extensively for medicinal purposes and as a disinfectant for small drinking-water supplies. Iodine, like bromine, is more costly than chlorine; although it is an effective disinfectant, it is usually too costly to use on a municipal scale. Due to possible health effects associated with long-term consumption, iodine is not recommended as a disinfectant for water supplies serving permanent populations. However, iodine can be used economically to disinfect water serving nonpermanent populations, as in campgrounds; and it can be used for emergency water disinfection. Iodine is available in crystals for use in saturator-type feeders.

**Ozon:** Ozone (03) is formed when a high-voltage arc passes through the air between two electrodes. It is also formed PHOTOCIIEMICALLY in the atmosphere and is one of the constituents in smog. Ozone is a bluish, toxic gas with a pungent odor. It is considered hazardous to health at a concentration in air of 0.25 mg/ L by volume and extremely hazardous at levels of 1.0 mg/ L and greater.

Ozone is a powerful oxidizing agent used in water treatment for disinfection and for control of color, taste, and odor. The gas is chemically unstable and disappears in minutes, leaving no residual disinfectant to continue disinfection within the water system. Furthermore, because of this lack of residual, the operator has no simple way to determine whether enough ozone has been added to destroy all pathogens. Bacteriological analysis of samples collected before and alter the point of application is the only method available for measurin2 the effectiveness of ozone.

Ozone has been used in a few US water plants for disinfection and taste-and- odor removal, but it has found use as a disinfectant primarily in Europe, mostly in France. Manufacturing ozone requires a great deal of energy, and rising energy costs are making ozonation more expensive to use. Chlorination, with its lower cost, greater flexibility, and greater familiarity, continues to be the preferred alternative in the United States.

Chlorine and chlorine compounds. Chlorine and chlorine compounds (such as calcium hypochlorite and sodium hypochlorite) are the disinfectant5 most widely used for water treatment in the United States, The remainder of this module describes chlorine and the chlorination of drinking water.



For example, if 2 mg/L of chlorine is added, 2mg/ L of free residual is produced. The reactions occurring are as follows

$$Cl_2 + H_2O \rightleftharpoons HOCl + H^+ + Cl^-$$

The products are weak compounds that dissociate as follows

$$HOCl \rightleftharpoons H^+ + OCl^-$$

Hypochlorous acid, one of two forms of free chlorine residual, is the most effective disinfectant available. When it dissociates as in Equation 53, the hypochlorite ion (the second form of free chlorine residual) is formed. The hypochlorite ion as a disinfectant is only 1/100 as effective as hypochlorous acid

| *Type* | *Chemical Abbreviation* | *Estimated Effectiveness Compared to HOCl* |
|---|---|---|
| Hypochlorous acid | HOCl | 1 |
| Hypochlorite ion | $OCl^-$ | 1/100 |
| Trichloramine* | $NCl_3$ | † |
| Dichloramine | $NHCl_2$ | 1/80 |
| Monochloramine | $NH_2Cl$ | 1/150 |

*Commonly called nitrogen trichloride.
†No estimate; possibly more effective than dichloramine.

Natural water is not pure, and the reaction of chlorine with the impurities it contains interferes with the formation of a free chlorine residual. For example, if the water contains organic matter, nitrites, iron, manganese, and ammonia, then the chlorine added will react. Between points l and 2. the chlorine added combines immediately with iron, manganese, and nitrites, iron, manganese, and nitrites are REDUCING AGENTS, and no residual can he formed until all reducing agents are completely destroyed by the chlorine.

As more chlorine is added between points 2 and 3, the chlorine begins to react with ammonia and organic matter to form chloramines and chloroorganic compounds. These are COMBINED CHLORINE RESIDUALS. Since the chlorine is combined with other compounds, this residual is not as effective as a free chlorine residual. (The effectiveness of residuals is compared)

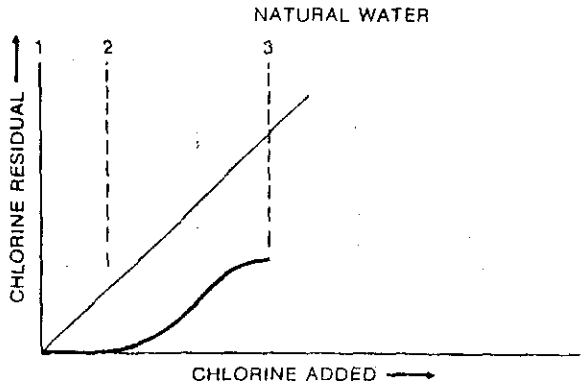



**Figure 10-3. The Reaction of Chlorine with Impurities in Natural Water**

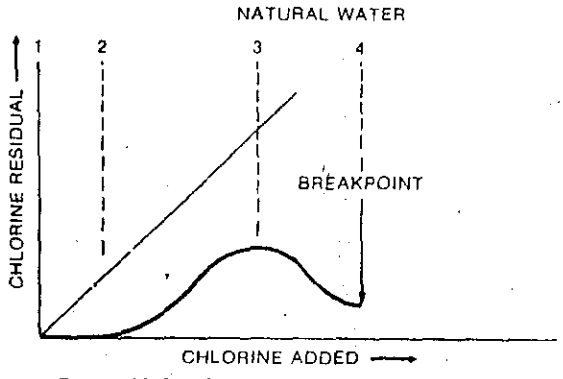



**Figure 10-4. Decrease of Chlorine Residual**

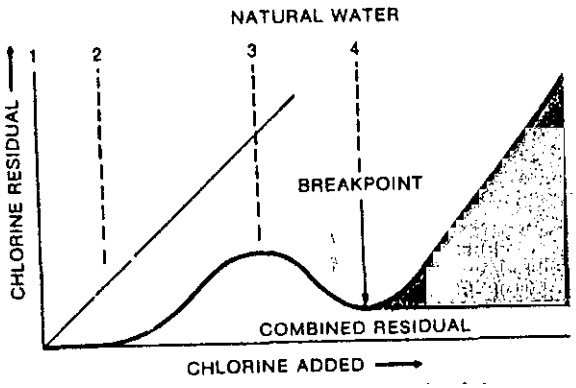
Between points 2 and 3, the combined residual is primarily monochloramine

$$NH_3 + HOCl \longrightarrow \underset{\text{(Monochloramine)}}{NH_2Cl} + H_2O$$

Adding more chlorine to the water actually decreases the residual. The decrease (shown from point 3 to point 4) results because the additional chlorine oxidizes some of the chloroorganic compounds and ammonia, The additional chlorine also changes some ofthe monochloraminc to dichlorarnine and trichloramine.

$$NH_2Cl + HOCl \longrightarrow \underset{\text{(Dichloramine)}}{NHCl_2} + H_2O \quad ($$

$$NHCl_2 + HOCl \longrightarrow \underset{\text{(Trichloramine or nitrogen trichloride}}{NCl_3} + H_2O \quad ($$




As additional chlorine is added, the amount of chloramine reaches a minimum value. Beyond this point, a point is reached where further addition of chlorine produces free residual chlorine. The point at which this occurs (point 4) is known as the BREAKPOINT. To the right of the breakpoint an increase in the chlorine dose will usually produce a proportionate increase in the free chlorine residual. Beyond the breakpoint, the free chlorine residual should be 85 to 90 percent of the total chlorine residual. The remaining 10 to 15 percent is combined residual consisting of dichloramines , trichloramines, and chloroorganic compounds. A group of the chloroorganic compounds, TRIHALOMETHANES

There are five factors important to the success of chlorination:

• Concentration

• Contact time

• Temperature

• pH

• Substances in the water.

**Concentration and contact time**: The effectiveness of chlorination depends primarily on two factors:

• Concentration (C)

• Contact time (t).

The destruction of organisms, often referred to as the "kill," is directly related to these two factors as follows:

**Kill *α* Concentration X Time**

This means that if the chlorine concentration is decreased, then the contact time —the length of time the chlorine and the organisms are in physical contact—must be increased to ensure that the kill remains the same Similarly. as the chlorine concentration increases, the contact time needed for a given kill decreases.

A combined chlorine residual, which is a weak disinfectant, requires a greater concentration acting over a longer period of time than is required for a cc chlorine residual. 'therefore, when the contact lime between the point ol application of chlorine and the consumption of water is short (for example, 0 mm), only a free residual will provide effective disinfection. It is important to know the contact time and type of residual chlorine available so the proper concentration can be provided. Usually, a minimum free chlorine residual of 0.5 mg/ L should be maintained at the extremities of the distribution system.

**Temperature.** The effectiveness of chlorination is also related to the temperature of the water—at lower temperature, bacterial kill tends to be slower. However, chlorine is more stable in cold water, and the residual will remain for a longer period of time, compensating to some extent for the lower .rate of disinfection. Other factors being equal, chlorination is more effective .with higher water temperatures. It is important for the operator to maintain a record of water temperatures. As temperatures change seasonally, the chlorine dosage will also need to be changed. The effectiveness of combined chlorine residuals is affected more by low temperatures than that of free chlorine

**pH**. The pH of the water affects the disinfecting action of chlorine since it determines the ratio of HOCL to OCL-—depending on the pH, either more hypochlorite ion or more hypochlorous acid could be present .the dissociation can go either direction, and the ratio of the ions will shift as the pH changes.

Hypochlorous acid dissociates poorly at low pH levels; the dominant residual is HOCl . On the other hand, HOCI will dissociate almost completely at high pH levels, leaving $0Cl^-$ as the dominant residual. The most dramatic dissociation occurs between pH 6.0 and pH 8.5, as shown in Figure. Note the slight effect of temperature on the distribution of HOCI and $OCl^-$at the various pH levels.. It is essential that the operator understand and use these relationships in order to obtain the most effective disinfectant. The operator should routinely check the pH. This is most important if the pH of the water is being raised to control corrosion, since the chlorine dosage will also have to be raised to maintain an effective level. Addition of chlorine gas lowers the pH of water; the use of hypochlorite raises the pH slightly.

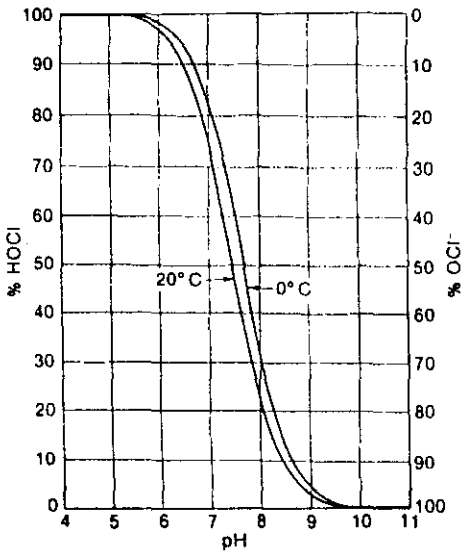



**Figure 10-7. Distribution of HOCl and $OCl^-$ in Water at Indicated pH Levels**

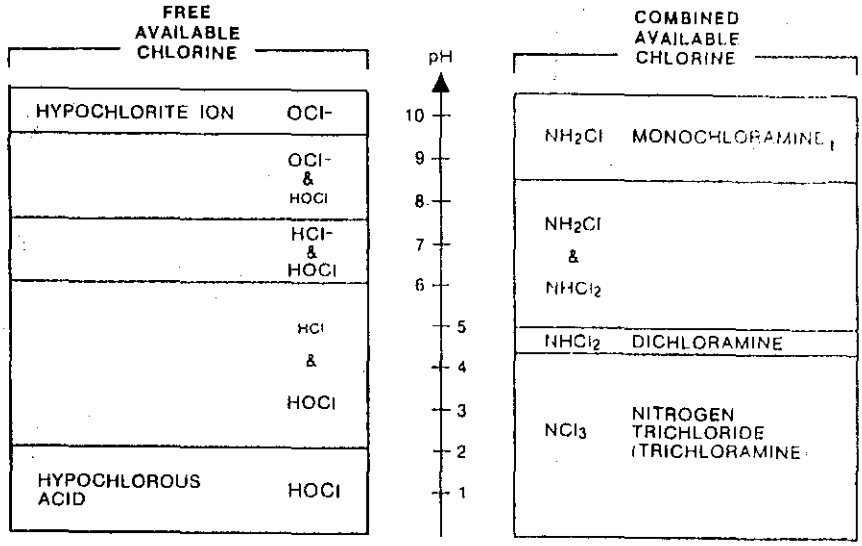



**Figure 10-8. Effect of pH on Free and Combined Chlorine Residual**

Substances in the water. Chlorine acts as an effective disinfectant only if it comes in contact with the organisms to be killed. Turbidity, caused by tiny particles of dirt and other impurities suspended in, water, can prevent good contact and protect the pathogens. Therefore, for chlorination to be effective. Turbidity must be reduced to the greatest extent possible by treatment methods such as coagulation, flocculation, and filtration.

As discussed earlier, chlorine reacts with other substances in water, such as organic matter and ammonia. Since these compounds result in the formation of the less—

effective combined residuals, their concentrations are an important factor in determining chlorine dosages. Breakpoint chlorination should be practiced to ensure the formation of free chlorine residual. Failing in practice breakpoint chlorination can allow the survival of pathogens and can cause serious taste-and-odor problems.

## Determining the Chlorine Dosage

The chlorine demand of the water being treated must be determined h performing a chlorine-demand test. Using the result from the demand test and knowing the residual needed, the operator can determine the chlorine dosage as follows

$$\text{Chlorine Dosage mg/L} = \text{Chlorine Demand mg/L} + \text{Chlorine Residual mg/L}$$

Once calculated, the dosage should be converted into pounds per day and the chlorinator should be set to deliver that dosage. After setting the dosage rate, the water must be tested regularly to ensure the proper chlorine residual is maintained.

As noted previously, temperature, pH and contact time are important variables affecting the success of chlorination. As these factors change, the amount of residual needed will also change.

**Table 10-7. Recommended Minimum Concentrations of Free Chlorine Residual Versus Combined Chlorine Residual to Ensure Effective Disinfection**

| *pH Value* | *Minimum Concentration of Free Chlorine Residual (disinfecting period is at least 10 min)* ppm | *Minimum Concentration of Combined Chlorine Residual (disinfecting period is at least 60 min)* ppm |
|---|---|---|
| 6.0–7.0 | 0.2 | 1.0 |
| 7.0–8.0 | 0.2 | 1.5 |
| 8.0–9.0 | 0.4 | 1.8 |
| 9.0–10.0 | 0.8 | Not recommended |
| 10.0+ | 0.8 + (with longer contact) | Not recommended |

as well as comparable values for combined residual. Free residual values are based on a contact time of at least 10 min the combined residual concentrations require at least 60 min contact time.

## Common Operating Problems

There are a variety of problems that can occur related to chlorination:

• Chlorine leaks

• Stiff container valves

• Tastes and odors

• Sudden change in residual

• Trihalotnethane formation.

Other problems can occur (particularly when using chlorine gas) since more piping, valves, and equipment arc involved. Proper maintenance can prevent many of these problems. Appendix B presents maintenance guidelines for gas chlorination equipment. If problems develop, the manufacturer's troubleshooting guides can be used to help locate and correct the problems.

### Chlorine Leaks

A major concern in the operation and maintenance of the chlorination process is the prevention of chlorine leaks. The most common place for leaks to occur is the pressurized chlorine supply line between the containers and the chlorinator. Every joint, valve, fitting, end gauge in the line is a possible point of leakage.

Some chlorine leaks are obvious. Others are very slow, very small, partly hidden, or otherwise difficult to locate. The usual method of detection is to open a bottle of ammonia solution, place the bottle near a suspected leak, and allow the ammonia fumes to rise around the suspect area. If there is a sizeable leak, the chlorine will combine with the ammonia to form visible, white, ammonium chloride vapor. Unfortunately, this method will not show the existence of a very small, slow leak (which also is usually too small to produce a noticeable odor).

Small leaks can go unnoticed for weeks unless the operators periodically look for two signs: (I) joint discoloration and (2) moisture.

Even the smallest leak will remove cadmium plating from chlorine tubing and fittings. The metal underneath (copper, brass, or bronze) will appear reddish, and a green copper-chloride scum may appear around the edges of i1he area affected.

Portions of the pressure piping system (the manifold, for example) are often painted. As a result, discoloration of the metal beneath the paint will not he apparent. To locate leaks in painted piping, look for small droplets of water that may form on the underside of joints. Small, almost-invisible leaks must he located early, or the corrosion they cause may in a matter of months or even weeks, result in a sudden and massive chlorine leak.

The best and most reliable way to detect chlorine leaks is with a chlorine detector. The detector is sensitive to very small leaks-—-as small as 1ppm chlorine in air. Such leaks are not normally detectable by the ammonia technique or by smell. If a major leak requires shutdown of the system for repair, then the tank valve should be closed, the yoke disconnected, and the injector left running with the auxiliary tank valve open until any remaining chlorine gas is purged from the line. Emergency kits for leak repair are

• Replace the lead gasket every time a cylinder or container is changed.

• Each time a threaded fitting is opened. clean the threads with a Wire brush and wrap with PTFE (sold tinder the trade name Teflon) tape or use one of the following pipe joint compounds: (I) linseed oil and graphite, (2) linseed oil and white lead, or (3) litharge and glycerin. If PTFE tape is used. remove any previous remnants of tape before remaking the joint.

• Replace all chlorine-supply-line valves annually. Refit and repack the old ones so they are ready for use the following year.

**Stiff Container Valves**

Container valves are carefully checked before leaving the manufacturer's plant, but occasionally a valve may be stiff to turn or difficult to shut off tightly. The problem is often caused by overly tight packing. Sometimes the valve can he freed by opening and shutting a few times. If the valve still does not operate. not the container aside and call the supplier.

**Taste and Odor**

There is a common 55understanding about the cause of "swimming pool" tastes and odors in drinking water. It seems only natural to assume that water that tastes like chlorine must contain too much chlorine. Oddly enough, chlorine-like tastes and odors are usually caused by too little chlorine, In the discussion of Figure it was pointed out that the space between points 2 and 4 is predominantly combined chlorine residual. The combined residual available in this range is the weakest form of chlorine for disinfection, and lie accompanying dichioramines. trichlnramines and chloroorgantc compounds cause taste and odor. By increasing the chlorine dosage beyond the breakpoint (beyond point 4 in Figure), the tastes- and odors are reduced significantly or eliminated, Free available chlorine residual is free from taste and, odor at the concentrations commonly used in disinfection.

**Sudden Change In Residual**

Chlorine offers a very valuable operating advantage over other disinfectant is:

measurable residual. Using a chlorine residual test procedure. the strength of disinfectant remaining in the water at any point in the distribution system can he determined. The test should take less than 5 min. Records of chlorine residual kept over several months or years can help predict the type and amount of residual that will be found at various locations in the distribution system. Any sudden drop in residual warns of a potentially o dangerous situation (for examplc a cross-connection that is allowing contaminated water into the drinking-water system).

When a sudden drop in residual occurs, chlorination levels should be increased immediately to bring the residual back to the desired level---this guards against the possibility of waterborne disease during the lime that the problem is being analyzed and corrected. Samples should then he analyzed to identify the contaminants and the distribution system should be checked locate the source of contamination or other cause of the drop in residual.

**Formation of Trihalomethanes**

All surface and ground wafers contain natural organic compounds. These compounds come from decaying vegetation add are primarily humic and fulvic acids. They can cause color, taste, and odor problems in water. In addition, when organic compounds react with chlorinee complex chloroorganic compounds known as Trihalomethane (THMs) are formed. These compounds. the most cotnmon of which is chloroform, are considered potential cancer-causing substances. Because of the possible public health effects, the USEPA has set a maximum contaminant level of 0.1 mg / L (100 ppb) for trihalornethanes in drinking water.

Once trihalometanes arc formed, they are very difficult to remove. Therefore, the best way to keep THM levels low is to prevent them from forming. Most of the trihalomcthanes are formed when a treated water is prechlorinated; the postchlortnation disinfection process is generally not the major contributor.

One means of reducing THMs levels is to discontinue prechlorination or move the prechlorination point to just ahead of the filters, This a lows the coagulation/flocculation/sedimentation processes to remove a large part of the humic compounds before they have a chance to react with chlorine. The disinfection process itself should not be altered until comprehensive monitoring and engineering studies have been conducted.

## Control Tests

Two types of operational testing are used to monitor the disinfection process. First, the level of chlorine residual is checked regularly at points throughout the distribution system and in the treatment plant. Second, bacteriological tests (for total coliform bacteria) are performed on samples from selected points in the distribution system.

Chlorine residual test. The chlorine residual test is essential to successful and efficient operation of the chlorination process. The test results provide the operator with three important pieces of information:

• Whether or not a residual exists

• The type of residual (free or combined)

• The amount of residual (concentration).

The operator should also monitor temperature and pH. These two factors influence the amount and type of residual formed, which in turn controls the effectiveness of disinfection.

The chlorine residual test is one of the quickest and easiest to run of all water plant operational control tests. The recommended test, the DI'D METHOD, can be performed using a field test kit, like any one of those shown in Figure 10-42. For laboratory use, the amperometric titrator is the most accurate method for measuring all forms of residual. The DPD method and the operational significance of chlorine residual are covered in detail in Volume 4, Introduction to Water Quality Analyses, Physical/Chemical Tests Module, Chlorine Demand and Residual.

As general rule, it is best to maintain a free available chlorine residual, since it is a far more effective disinfectant than a combined residual. Usually, a free residual ofO.3 mg/L is the minimum level to maintain at the extremities of the distribution system. The chlorinator dosage must be set to ensure that the desired level of free residual will exist at all times in all parts of the system.

Bacteriological test. All pathogenic organisms have a common source—the feces of humans and animals. One obvious way to determine the success of disinfection would be to test the treated water for the presence of pathogens. However, this is not practical since pathogens in water are few in number and quite difficult to measure, even with sophisticated laboratory equipment.

Fortunately. there is a group of bacteria, known as coliform bacteria, which are relatively easy to measure and whose presence indicates that pathogens might also be present. Since most pathogens are less resistant to chlorine than coliform bacteria, it is assumed that if no coliform bacteria are found in treated water, pathogens are not present. Therefore, routine sample are taken from the raw and treated water (including the distribution system) and tested for coliform bacteria (total coliform test).

# MORE EXPLANATIONS FOR CHLORINE CHEMISTRY

## Chlorination Materials

### Elemental Chlorine

**Physical properties.** In the gaseous state, chlorine is greenish-yellow in color and is about 2 1/2 times as heavy as air. In commerce, it is available as a compressed liquid that is amber colored and about 1 1/2 times as heavy as water. If liquid chlorine is unconfined, it rapidly vaporizes to gas (one volume of liquid yields about 450 volumes of gas). Since chlorine confined in a container may exist as a gas, as a liquid, or as both, any consideration of liquid chlorine includes that of gaseous chlorine. As will be noted later, gas vapor pressure of chlorine in a container is a function of temperature and is independent of the contained volume of chlorine. Thus, gage pressure of a container having 1 lb of chlorine will be essentially the same as if it contained 100 lbs of chlorine (under the same temperature conditions). Note that when the chlorine temperature increases so does the pressure. Note that a shipping container filled to its authorized limit becomes completely full of liquid at approximately 154F. Temperatures beyond that point may produce pressures that could result in hydrostatic rupture of the container. Safety devices always are provided to relieve excessive pressures accompanying dangerous temperature increases.

Chlorine is only slightly soluble in water; its maximum solubility is approximately 1 per cent at 49.2C. At temperatures below this point, it combines with water to form chlorine "ice" — a crystalline substance. (When the water supplying a chlorinator is at a sufficiently low temperature to cool moist chlorine gas to the point at which chlorine ice is formed, accumulations on the needle valve and gas outlet tube develop and erratic feed results; this condition can be corrected by slightly warming either the portion of the water that comes in contact with the chlorine or the gas feed line.) Because the vapor pressure of chlorine increases with rising temperature, its solubility also decreases. At 212F chlorine is insoluble in water; between 49.2F and 212F, chlorine dissolved in water forms a weak, corrosive mixture of hydrochloric and hypochlorous acids. The corrosivity of chlorine-water solutions creates problems in handling.

**Chemical properties.** Under specific conditions chlorine reacts with most elements, sometimes extremely rapidly. Because of its great affinity for hydrogen, chlorine removes hydrogen from some of its compounds, such as in the reaction with hydrogen sulfide. It also reacts, as later discussed, with ammonia or other nitrogen-containing compounds to form various mixtures of chloramines. It reacts much the same with organic materials, sometimes with explosive violence.

Although it is neither explosive nor flammable, chlorine (like oxygen) iscapable of supporting combustion of certain substances. It should, therefore, be handled and stored away from other compressed gases (e.g., ammonia) and flammable materials. Most common metals are not affected at normal temperatures by *dry* chlorine, either gas or liquid. Chlorine is, however, reactive with aluminum and ignites carbon steel at temperatures above about 450F. Moist chlorine is corrosive to all common metals with the exception of gold, silver, platinum, titanium, and certain specialized alloys.

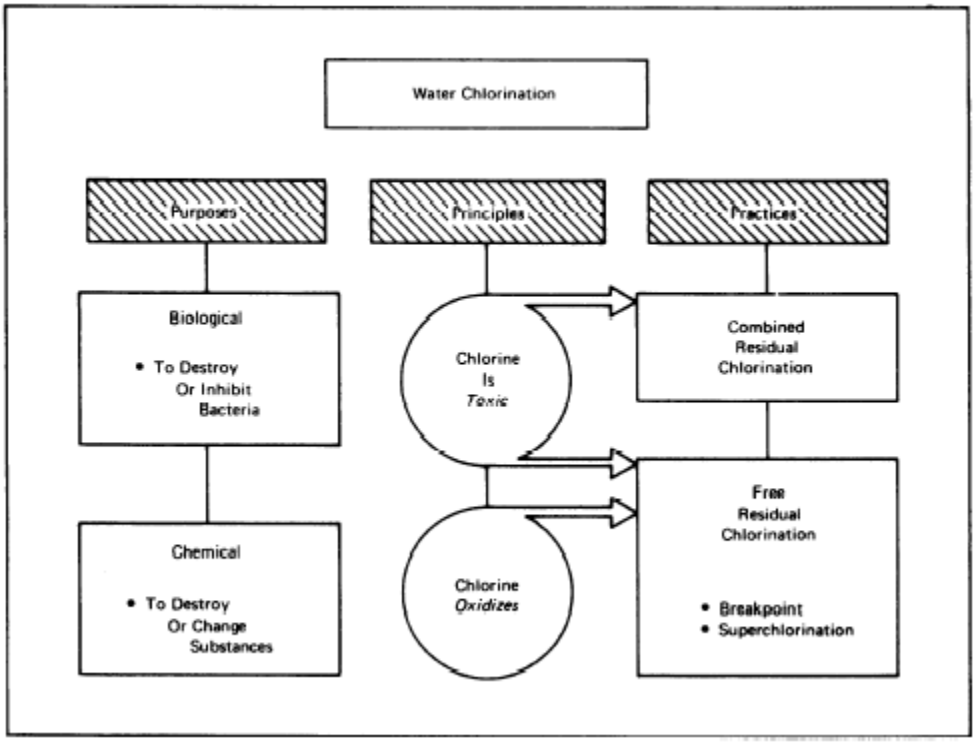
**Physiological effects.** Chlorine gas is primarily a respiratory irritant, and concentrations in air above 3 ppm by volume can usually be detected (by smell) by most persons.

Chlorine causes varying degrees of irritation of the skin (particularly in the presence of moisture), mucus membranes, and the respiratory system, depending on the concentration and duration of exposure. In severe cases death can occur from suffocation. However, the extremely irritating effect of the gas makes it unlikely that any person would deliberately remain in a dangerous atmosphere unless he were unconscious or trapped.

Liquid chlorine may cause skin and eye bums upon contact with these tissues. (When unconfined in a container, chlorine vaporizes rapidly, producing the
same effects as the gas.)

Chlorine produces no known cumulative effects and complete recovery usually can be expected to occur shortly following mild, short-term acute exposures. An 8-hr, time-weighted exposure of up to 1 ppm (approximately 3 mg/cu m) is the current national consensus standard provided for in the 1971 Occupational Safety and Health Act. Concentrations of 40-60 ppm for 30-60 min are dangerous, and higher concentrations, even for very brief periods, can be fatal.

Chlorine is used in water treatment (1) for disinfection — to destroy or "inactivate" microorganisms of sanitary significance, (2) for oxidation — to purposefully modify the chemical character of the water to which it is applied, or (3) for both disinfection and oxidation. Actually, the toxic character of chlorine and some of its compounds is due in large measure to their oxidizing




capacity, but oxidizing capacity is not necessarily a measure of germicidal efficiency. Disinfection is the chief and most common objective of chlorination. In fact, the terms disinfection and chlorination largely have come to imply the same treatment. As discussed later, not all chlorine forms, or their reaction products

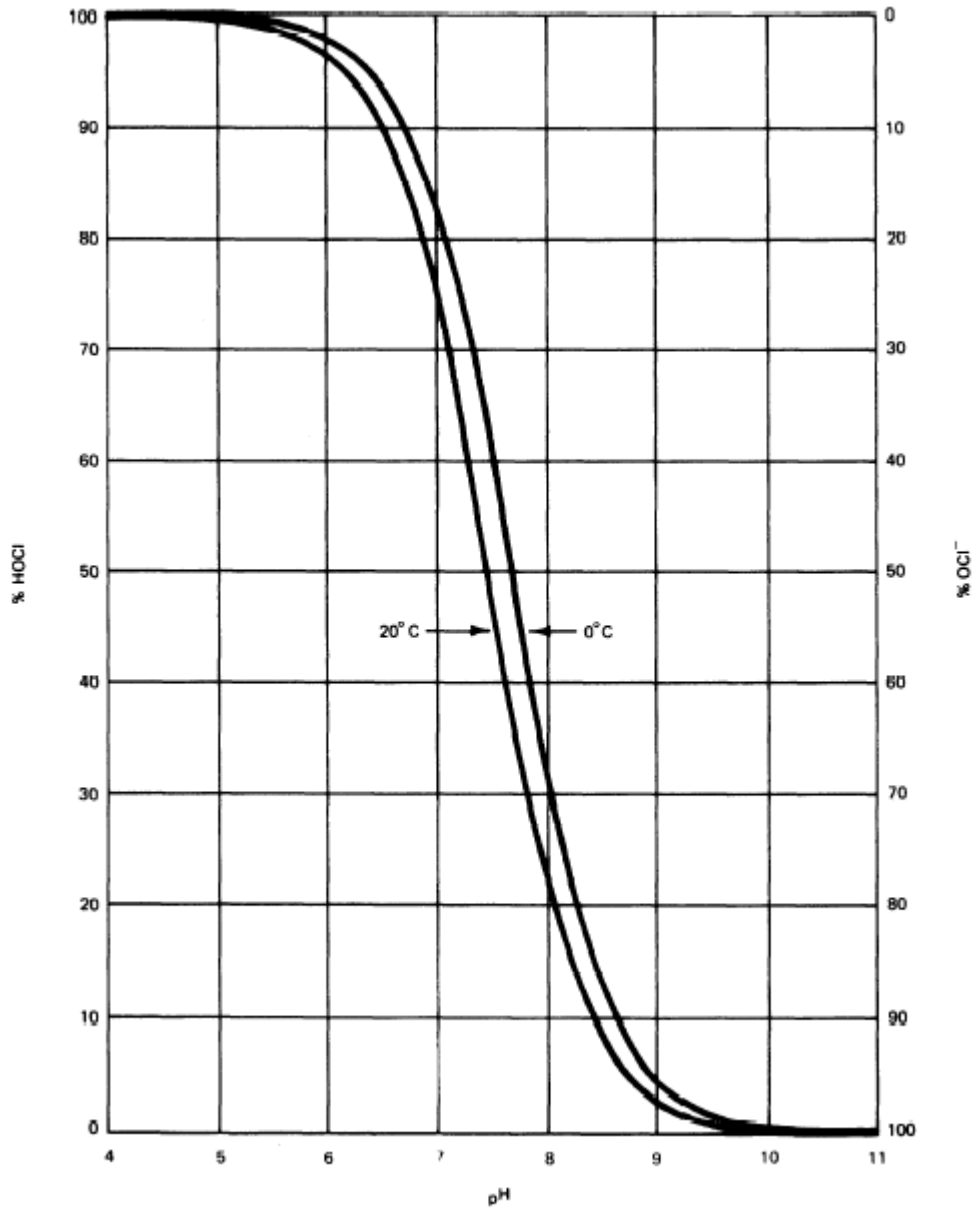



**Fig. 4.** Distribution of HOC1 and $OC1^-$ in Water at Indicated pH Levels

## Reactions With Water

When chlorine is added to chemically pure water, a mixture of hypochlorous (HOC1) and hydrochloric (HC1) acids is formed: a highly efficient one, due largely to improved methods of testing the effectiveness of treatment and, through these, to improved definition of the biological and chemical processes involved.

$$Cl_2 + H_2O \rightleftharpoons HOCl + H^+ + Cl^- \qquad \text{(Eq Z–2)}$$

At ordinary water temperatures this reaction is essentially complete within a few seconds. In dilute solution and at pH levels above about 4, the equilibrium shown in Eq 2-2 is displaced to the right and very little $C1_2$ exists in solution. In normal practice (except for the concentrate from solution-feed chlorinators), the amount of chlorine supplied to water does not produce a concentrated solution of such strength as to yield such a low pH. The oxidizing property of the chlorine is, however, retained in the HOC1 formed, and it is also in this form that the principal disinfecting action of aqueous chlorine solutions occurs.

Hypochlorous acid ionizes or dissociates, a practically instantaneous reaction, into hydrogen and hypochlorite ions (note that this reaction also is reversible), the degree of dissociation depending on pH and temperature:

$$HOCl \rightleftharpoons H^+ + OCl^- \qquad \text{(Eq Z–3)}$$

Hypochlorous acid is weak and dissociates poorly at levels of pH below about 6; thus, chlorine exists predominantly as HOC1 at relatively low pH levels. Between pH 6.0 and 8.5 there occurs a very sharp change from undissociated HOC1 to almost complete dissociation. At 20C above about pH 7.5 and at 0C above about pH 7.8, hypochlorite ions OC1–) predominate, and they exist almost exclusively at levels of pH around 9.5 and above . The pH of chlorinated water supplies is normally within the range where chlorine may exist as both hypochlorous acid and hypochlorite ion. *Chlorine existing in water as hypochlorous acid and hypochlorite ions is defined as free available chlorine.*

Chlorine-containing compounds, such as high-test (70 per cent available chlorine) calcium hypochlorite, $Ca(OC1)_2$, and sodium hypochlorite, NaOCl, ionize in water and also yield hypochlorite ions:

$$Ca(OCl)_2 + H_2O \rightleftharpoons Ca^{++} + 2OCl^- + H_2O \qquad \text{(Eq Z–4)}$$

$$NaOCl + H_2O \rightleftharpoons Na^+ + OCl^- + H_2O \qquad \text{(Eq Z–5)}$$

The hypochlorite ions also establish equilibrium with hydrogen ions, depending on the pH, as shown by Eq Z-3. Thus, the same equilibria are established in water regardless of whether gaseous chlorine or hypochlorite chlorine compounds are employed. The important distinction is the resultant pH and hence the relative amounts of HOC1 and $OC1^-$ existing at equilibrium. Chlorine tends to decrease the initial pH, and hypochlorites tend to increase it. From Eq Z-2, it is evident that the hypochlorous and hydrochloric acids produced tend to lower pH and reduce alkalinity. Each part of chlorine added will neutralize not less than 0.7 part of alkalinity (as $CaCO_3$), and may neutralize 1.4 parts, depending upon the extent to which HOC1 is ionized and the manner in which chlorine is consumed by substances in the water. Hardness producing materials, principally the carbonates and bicarbonates of calcium and magnesium, tend to buffer water against significant changes in pH that otherwise would accompany the addition of small amounts of acid or alkaline materials. If sufficient amounts of chlorine are applied to deplete natural alkalinity to such an extent that the pH is substantially decreased, however, pH correction may be indicated as auxiliary treatment. Hypochlorites always contain excess alkali for stability and tend to raise pH levels, but usually by insignificant amounts.

**Chlorine-Ammonia Reactions**

The reactions of chlorine with inorganic nitrogenous material, such as ammonia, are of great significance in water chlorination processes, especially disinfection, The kinetics of these reactions have been extensively studied. When chlorine is added to water containing natural or added ammonia (ammonium ion exists in equilibrium with ammonia and hydrogen ions), the ammonia reacts with HOC1 to form various chloramines that, like HOC1, retain the oxidizing power of the chlorine. The reactions between chlorine and ammonia may be represented by

$$NH_3 + HOCl \longrightarrow NH_2Cl + H_2O \quad \text{(Eq Z–6)}$$
(Monochloramine)

$$NH_2Cl + HOCl \longrightarrow NHCl_2 + H_2O \quad \text{(Eq Z–7)}$$
(Dichloramine)

$$NHCl_2 + HOCl \longrightarrow NCl_3 + H_2O \quad \text{(Eq Z–8)}$$
(Trichloramine or nitrogen trichloride)

The distribution of the reaction products is governed by the rates of formation of monochloramine and dichloramine, which are dependent upon pH, temperature, time, and initial C1:$NH_3$ ratio. In general, high C1:$NH_3$ ratios, low temperatures, and low pH levels favor dichloramine formation. Figure 5 illustrates the relative amounts of monochloramine and dichloramine likely to be formed at various pH levels when the C1:$NH_3$ ratio is equimolar (5:1 on a weight basis). Although reported data differ to some degree, it is evident that some dichloramine can be anticipated at pH levels below 7. Other work further suggests that at pH levels below 7.5, some odorous nitrogen trichloride also can be anticipated.

Chlorine also reacts with organic nitrogenous materials, such as proteins and amino acids, to form organic chloramine complexes. *Chlorine existing in water in chemical combination with ammonia-nitrogen or organic nitrogen compounds is defined as combined available chlorine.*

The oxidizing capacity of free chlorine solutions varies with pH because of variations in the resultant HOC1:$OC1^-$ ratios, decreasing at high pH levels where $OC1^-$ predominates. This principle applies also in the case of chloramines solutions as a result of varying $NHC1_2$:$NH_2C1$ ratios, and where monochloramine predominates at high pH levels. Chloramines are less potent disinfectants and are less reactive than free available chlorine forms.

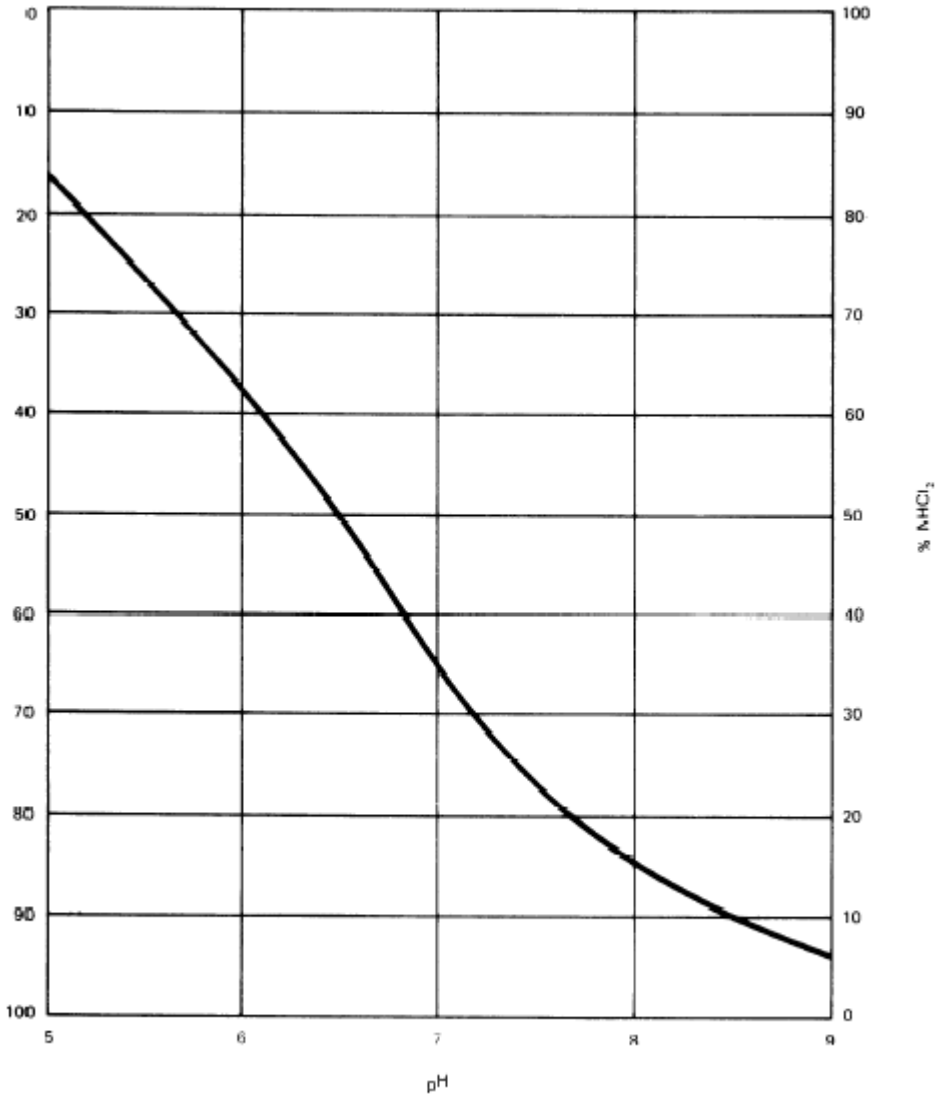
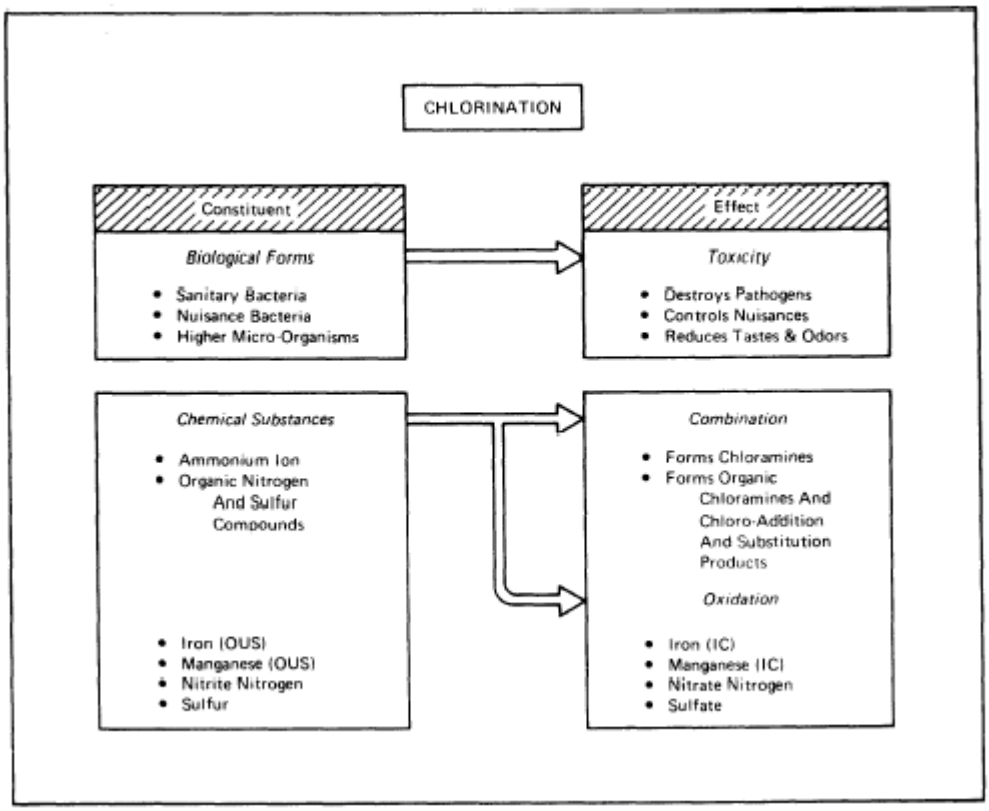



**Fig. 5.** Distribution of Chloramines in 25C Water at Indicated pH Levels and $Cl_2$: $NH_3$ (as N) Weight Ratio 5:1




**Iron and manganese** (Fe and Mn) in water also affect chlorination practices; if the pH is high enough for hydroxide formation and if sufficient chlorine is added, ferrous and manganous forms of the metals will be rapidly oxidized to insoluble hydroxide forms. Chloramines do not enter vigorously into these oxidation reactions. For satisfactory manganous oxidation, free available chlorine, as distinct from combined available chlorine, is required. For iron removal the optimum pH is 7 or above, and for manganese removal the optimum pH is 10. About 0.6 and 1.3 parts of chlorine are required, respectively, for each part of Fe and Mn removed, and about 0.9 and 3.4 parts of alkalinity (as $CaCO_3$) are consumed.

**Nitrites** are occasionally present in some waters, particularly those contaminated by domestic and certain industrial wastes. They, too, are readily oxidized to nitrates by chlorine, particularly free available chlorine, each part of nitrite consuming about 1.5 parts of chlorine.

**A few organic reducing** materials may be present in potable waters but their concentration usually is low. However, there are a variety of chlorine-reactive, organic

materials in some waters that can exert a substantial influence on chlorine requirements, depending upon available chlorine concentration and reaction time. Some dissolved organic materials react with chlorine rapidly, but, generally, the time required for the completion of most organic reactions is a matter of hours. Under some conditions, chlor-addition or chlor-substitution products might be formed, and under others the organic material may be completely oxidized. (Organic chlorine complexes generally have low bactericidal capacity and are of principal concern in water-chlorination practice because of the chlorine consumed and the tastes and odors they may impart.) Biological and chemical effects of chlorine in water are illustrated in Fig. 6.

## Chlorine Generation

### *Chlorine*

Chlorine gas can be generated by a number of processes including the electrolysis of alkaline brine or hydrochloric acid, the reaction between sodium chloride and nitric acid, or the oxidation of hydrochloric acid. About 70 percent of the chlorine produced in the United States is manufactured from the electrolysis of salt brine and caustic solutions in a diaphragm cell. Since chlorine is a stable compound, it is typically produced off-site by a chemical manufacturer. Once produced, chlorine is packaged as a liquefied gas under pressure for delivery to the site in railcars, tanker trucks, or cylinders.

### *Sodium Hypochlorite*

Dilute sodium hypochlorite solutions (less than 1 percent) can be generated electrochemically on-site from salt brine solution. Typically, sodium hypochlorite solutions are referred to as liquid bleach or Javelle water. Generally, the commercial or industrial grade solutions produced have hypochlorite strengths of 10 to 16 percent. The stability of sodium hypochlorite solution depends on the hypochlorite concentration, the storage temperature, the length of storage (time), the impurities of the solution, and exposure to light. Decomposition of hypochlorite over time can affect the feed rate and dosage, as well as produce undesirable byproducts such as chlorite ions or chlorate.

Because of the storage problems, many systems are investigating onsite generation of hypochlorite in lieu of its purchase from a manufacturer or vendor.

### *Calcium Hypochlorite*

To produce calcium hypochlorite, hypochlorous acid is made by adding chlorine monoxide to water and then neutralizing it with a lime slurry to create a solution of calcium hypochlorite. The water is removed from the solution, leaving granulated calcium hypochlorite. Generally, the final product contains up to 70 percent available chlorine and 4 to 6 percent lime. Storage of calcium hypochlorite is a major safety consideration. It should never be stored where it is subject to heat or allowed to contact any organic material of an easily oxidized nature .

**Table 2-18. Chlorine Uses and Doses**

| Application | Typical Dose | Optimal pH | Reaction Time | Effectiveness | Other Considerations |
|---|---|---|---|---|---|
| Iron | 0.62 mg/mg Fe | 7.0 | less than 1 hour | Good | |
| Manganese | 0.77 mg/mg Mn | 7–8<br>9.5 | 1–3 hour<br>minutes | Slow kinetics | Reaction time increases at lower pH |
| Biological growth | 1–2 mg/L | 6–8 | NA | Good | DBP formation |
| Taste/odor | Varies | 6–8 | Varies | Varies | Effectiveness depends on compound |
| Color removal | Varies | 4.0–6.8 | Minutes | Good | DBP formation |
| Zebra mussels | 2–5 mg/L<br>0.2–0.5 mg/L[(a)] | | Shock level<br>Maintenance level | Good | DBP formation |
| Asiatic clams | 0.3–0.5 mg/L[(a)] | | Continuous | Good | DBP formation |

Notes:
[(a)] Residual, not dose

Sources: Adapted in part from White, 1992; Connell, 1996; Culp/Wesner/Culp, 1986.

### *Points of Application*

At conventional surface water treatment plants, chlorine is typically added for prechlorination at either the raw water intake or flash mixer, for intermediate chlorination ahead of the filters, for postchlorination at the filter clearwell, or for rechlorination of the distribution system . Table  summarizes the typical uses for each point of application.

Table 2-19. Typical Chlorine Points of Application and Uses

| Point of Application | Typical Uses |
|---|---|
| Raw Water Intake | Zebra mussel and Asiatic clam control, control biological growth |
| Flash Mixer (prior to sedimentation) | Disinfection, iron and manganese oxidation, taste and odor control, oxidation of hydrogen sulfide |
| Filter Influent | Disinfection, control biological growth in filter, iron and manganese oxidation, taste and odor control, algae control, color removal |
| Filter Clearwell | Disinfection |
| Distribution System | Maintain disinfectant residual |

Sources: Connell, 1996; White, 1992; AWWA, 1990.

### ***Typical Doses***

Table  shows the typical dosages for the various forms of chlorine. The wide range of chlorine gas dosages most likely represents its use as both an oxidant and a disinfectant. While sodium hypochlorite and calcium hypochlorite can also serve as both an oxidant and a disinfectant, their higher cost may limit their use.

Table 2-20. Typical Chlorine Dosages at Water Treatment Plants

| Chlorine Compound | Range of Doses |
|---|---|
| Calcium hypochlorite | 0.5–5 mg/L |
| Sodium hypochlorite | 0.2–2 mg/L |
| Chlorine gas | 1–16 mg/L |

Source: SAIC, 1998, as adapted from EPA's review of public water systems' Initial Sampling Plans which were required by EPA's Information Collection Rule (ICR)

### *Environmental Effects*

Several environmental factors influence the inactivation efficiency of chlorine, including water temperature, pH, contact time, mixing, turbidity, interfering substances, and the concentration of available chlorine. In general, the highest levels of pathogen inactivation are achieved with high chlorine residuals, long contact times, high water temperature, and good mixing, combined with a low pH, low turbidity, and the absence of interfering substances. Of the environmental factors, pH and temperature have the most impact on pathogen inactivation by chlorine. The effect of pH and temperature on pathogen inactivation are discussed below.

**pH.** The germicidal efficiency of hypochlorous acid (HOCl) is much higher than that of the hypochlorite ion ($OCl^-$). The distribution of chlorine species between HOCl and $OCl^-$ is determined by pH, as discussed above. Because HOCl dominates at low pH,

chlorination provides more effective disinfection at low pH. At high pH, $OCl^-$ dominates, which causes a decrease in disinfection efficiency.

The inactivation efficiency of gaseous chlorine and hypochlorite is the same at the same pH after chlorine addition. Note, however, that addition of gaseous chlorine will decrease the pH while the addition of hypochlorite will increase the pH of the water . Therefore, without pH adjustment to maintain the same treated water pH, gaseous chlorine will have greater disinfection efficiency than hypochlorite.

The impact of pH on chlorine disinfection has been demonstrated in the field. For example, virus inactivation studies have shown that 50 percent more contact time is required at pH 7.0 than at pH 6.0 to achieve comparable levels of inactivation. These studies also demonstrated that a rise in pH from 7.0 to 8.8 or 9.0 requires six times the contact time to achieve the same level of virus inactivation . Although these studies found a decrease in inactivation with increasing pH, some studies have shown the opposite effect. A 1972 study reported that viruses were more sensitive to free chlorine at high pH than at low pH .

**Temperature**. For typical drinking water treatment temperatures, pathogen inactivation increases with temperature. Virus studies indicate that the contact time should be increased by two to three times to achieve comparable inactivation levels when the water temperature is lowered by $10^oC$ .

## Pathogen Inactivation and Disinfection Efficacy

### *Inactivation Mechanisms*

Research has shown that chlorine is capable of producing lethal events at or near the cell membrane as well as affecting DNA. In bacteria, chlorine was found to adversely affect cell respiration, transport, and possibly DNA activity. Chlorination was found to cause an immediate decrease in oxygen utilization in both Escherichia coli and Candida parapsilosis studies. The results also found that chlorine damages the cell wall membrane, promotes leakage through the cell membrane, and produces lower levels of DNA synthesis for Escherichia coli, Candida parapsilosis, and Mycobacterium fortuitum bacteria. This study also showed that chlorine inactivation is rapid and does not require

bacteria reproduction. These observations rule out mutation or lesions as the principal inactivation mechanisms since these mechanisms require at least one generation of replication for inactivation to occur.

## Operational Considerations

### *Application Methods*

Different application methods are used, depending upon the form of chlorine used. The following paragraphs describe the typical application methods for chlorine, sodium hypochlorite, and calcium hypochlorite.

**Chlorine.** Liquefied chlorine gas is typically evaporated to gaseous chlorine prior to metering. The heat required for evaporation can be provided through either a liquid chlorine evaporator or the ambient heat input to the storage container. Once the compressed liquid chlorine is evaporated, chlorine gas is typically fed under vacuum conditions. Either an injector or a vacuum induction mixer usually creates the required vacuum. The injector uses water flowing through a venturi to draw the chlorine gas into a side stream of carrier water to form a concentrated chlorine solution. This solution is then introduced into the process water through a diffuser or mixed with a mechanical mixer. A vacuum induction mixer uses the motive forces of the mixer to create a vacuum and draws the chlorine gas directly into the process water at the mixer.

**Sodium Hypochlorite.** Sodium hypochlorite solutions degrade over time. For example, a 12.5 percent hypochlorite solution will degrade to 10 percent in 30 days under "best case" conditions . Increased temperature, exposure to light, and contact with metals increase the rate of sodium hypochlorite degradation .

Sodium hypochlorite solution is typically fed directly into the process water using a type of metering pump. Similar to chlorine solution, sodium hypochlorite is mixed with the process water with either a mechanical mixer or induction mixer. Sodium hypochlorite solution is typically not diluted prior to mixing to reduce scaling problems.

**Calcium Hypochlorite**.

Commercial high-level calcium hypochlorite contains at least 70% available chlorine . Under normal storage conditions, calcium hypochlorite loses 3 to 5% of its available chlorine in a year . Calcium hypochlorite comes in powder, granular, and compressed tablet forms. Typically, calcium hypochlorite solution is prepared by mixing powdered or granular calcium hypochlorite with a small flow. The highly chlorinated solution is then flow paced into drinking water flow.

# CT Factor

One of the most important factors for determining or predicting the germicidal efficiency of any disinfectant is the CT factor

The CT factor is defined as the product of the residual disinfectant concentration, C, in mg/L, and the contact time, T, in minutes, that residual disinfectant is in contact with the water.

EPA developed CT values for the inactivation of Giardia and viruses under the SWTR. Table  compares the CT values for virus inactivation using chlorine, chlorine dioxide, ozone, chloramine, and ultraviolet light disinfection under specified conditions. Table  shows the CT values for inactivation of Giardia cyst using chlorine, chloramine, chlorine dioxide, and ozone under specified conditions. The CT values shown in Table 2-13 and Table 2-14 are based on water temperatures of 10°C and pH values in the range of 6 to 9. CT values for chlorine disinfection are based on a free chlorine residual. Note that chlorine is less effective as pH increases from 6 to 9. In addition, for a given CT value, a low C and a high T is more effective than the reverse .

For all disinfectants, as temperature increases, effectiveness increases.

**Table 2-13. CT Values for Inactivation of Viruses**

| Disinfectant | Units | Inactivation | | |
|---|---|---|---|---|
| | | 2-log | 3-log | 4-log |
| Chlorine[1] | mg · min/L | 3 | 4 | 6 |
| Chloramine[2] | mg · min/L | 643 | 1,067 | 1,491 |
| Chlorine Dioxide[3] | mg · min/L | 4.2 | 12.8 | 25.1 |
| Ozone | mg · min/L | 0.5 | 0.8 | 1.0 |
| UV | $mW \cdot s/cm^2$ | 21 | 36 | not available |

CT values were obtained from AWWA, 1991.
[1] Values are based on a temperature of 10°C, pH range of 6 to 9, and a free chlorine residual of 0.2 to 0.5 mg/L.
[2] Values are based on a temperature of 10°C and a pH of 8.
[3] Values are based on a temperature of 10°C and a pH range of 6 to 9.

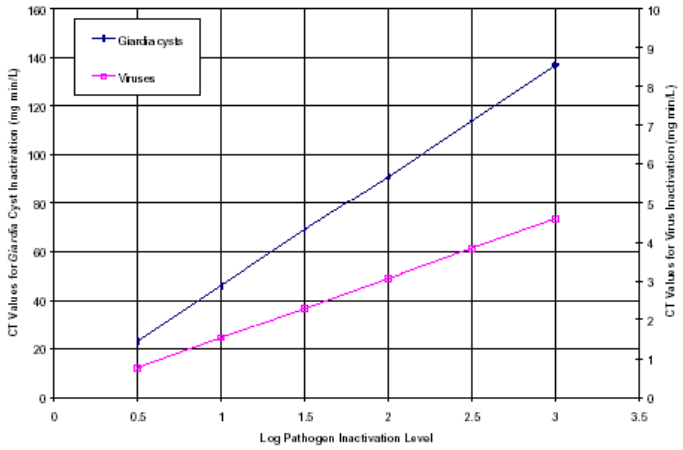
**Table 2-14. CT Values for Inactivation of *Giardia* Cysts**

| Disinfectant | Inactivation (mg · min/L) | | | | | |
|---|---|---|---|---|---|---|
| | 0.5-log | 1-log | 1.5-log | 2-log | 2.5-log | 3-log |
| Chlorine[1] | 17 | 35 | 52 | 69 | 87 | 104 |
| Chloramine[2] | 310 | 615 | 930 | 1,230 | 1,540 | 1,850 |
| Chlorine Dioxide[3] | 4 | 7.7 | 12 | 15 | 19 | 23 |
| Ozone[3] | 0.23 | 0.48 | 0.72 | 0.95 | 1.2 | 1.43 |

CT values were obtained from AWWA, 1991.
[1] Values are based on a free chlorine residual less than or equal to 0.4 mg/L, temperature of 10°C, and a pH of 7.
[2] Values are based on a temperature of 10°C and a pH in the range of 6 to 9.
[3] Values are based on a temperature of 10°C and a pH of 6 to 9.

### *CT Curves*

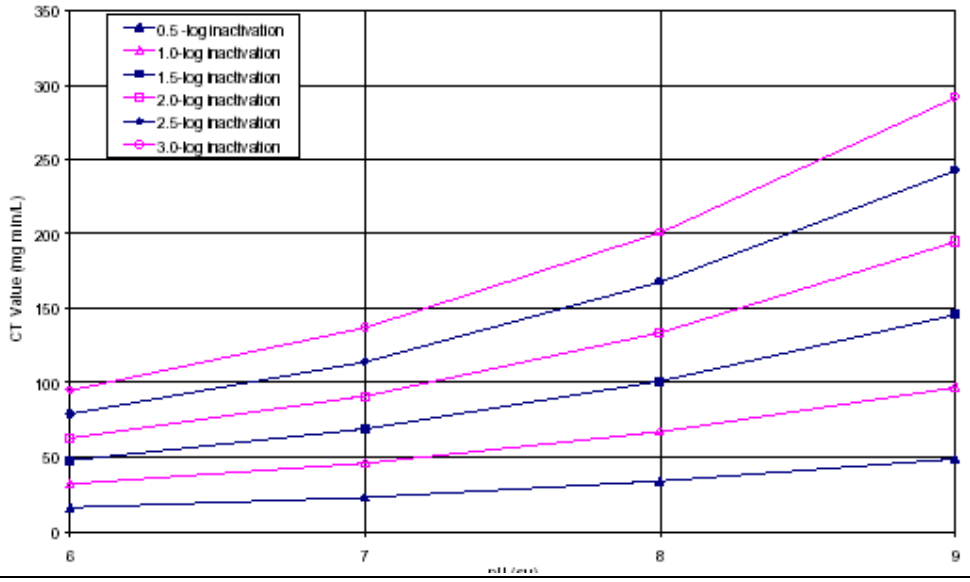
Chlorine is regarded as a strong disinfectant that is effective at inactivating bacteria and viruses, and under certain circumstances, Giardia. Because of chlorine's extremely high virus inactivation efficiency, CT values are almost always governed by protozoa inactivation. For example, Figure 2-1 shows the CT values required to achieve between 0.5 and 3-logs of virus and Giardia inactivation . As shown, the CT values required to achieve the recommended disinfection efficiency for conventional filtration systems are 23 and 3 mg min/L, respectively.



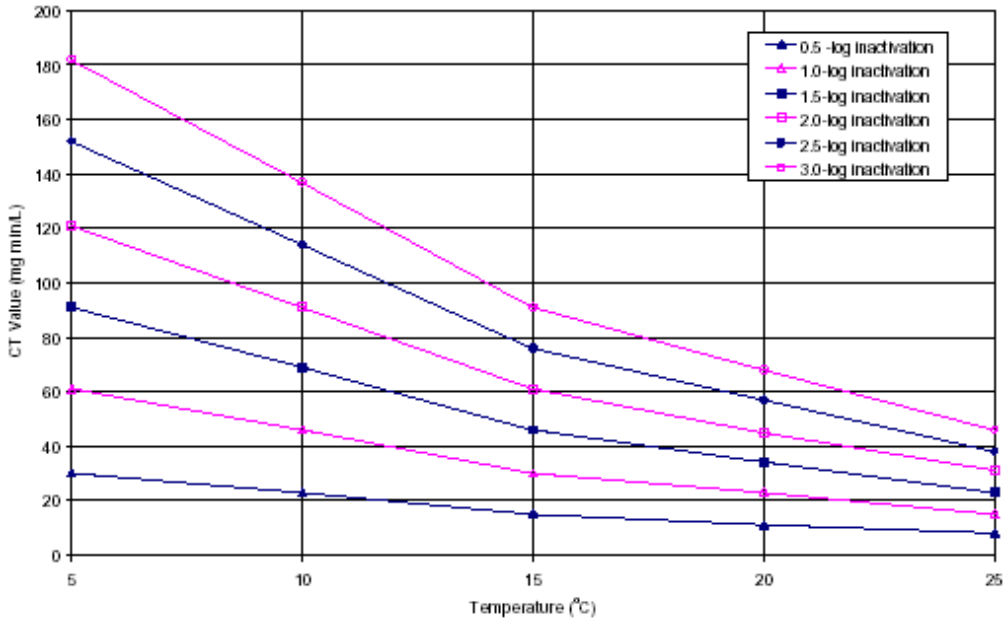

**Free Chlorine *Giardia* and Virus CT Requirements**

**Free Chlorine *Giardia* and Virus CT Requirements**

CT values for Giardia inactivation for various pH values and temperatures at a chlorine dose of 3.0 mg/L are shown in Figures . As shown, the inactivation efficacy of free chlorine decreases with increasing pH and/or decreasing temperature. CT values shown in Figures 2-2 and 2- 3 are based on animal infectivity and excystation studies. CT values ranging from 0.5 to 3-log inactivation at temperatures of 0.5 and 5oC were based on a multiplicative model, and applying first order kinetics to the 99 percent upper confidence interval of the 99.99-percentile CT values. CT values for temperatures above 5$^{o}$C were estimated by assuming a twofold decrease for every 10$^{o}$C decrease in temperature.




**CT Values for Inactivation of *Giardia* Cysts by Free Chlorine at 10₀C (at $Cl_2$ dose of 3.0 mg/L)**




**CT Values for Inactivation of *Giardia* Cysts by Free Chlorine at pH 7.0 (at $Cl_2$ dose of 3.0 mg/L)**

# Disinfection Byproducts and Disinfection Residuals

## Types of DBPs and Disinfection Residuals

Table is a list, of DBPs and disinfection residuals that may be of health concern. The table includes both the disinfectant residuals and the specific byproducts produced by the disinfectants of interest in drinking water treatment. These contaminants of concern are grouped into four distinct categories and include disinfectant residuals, inorganic byproducts, organic oxidation byproducts, and halogenated organic byproducts.

**Table 2-8. List of Disinfection Byproducts and Disinfection Residuals**

| DISINFECTANT RESIDUALS | HALOGENATED ORGANIC BYPRODUCTS |
|---|---|
| Free Chlorine | Trihalomethanes |
| Hypochlorous Acid | Chloroform |
| Hypochlorite Ion | Bromodichloromethane |
| Chloramines | Dibromochloromethane |
| Monochloramine | Bromoform |
| Chlorine Dioxide | Haloacetic Acids |
| **INORGANIC BYPRODUCTS** | Monochloroacetic Acid |
| Chlorate Ion | Dichloroacetic Acid |
| Chlorite Ion | Trichloroacetic Acid |
| Bromate Ion | Monobromoacetic Acid |
| Iodate Ion | Dibromoacetic Acid |
| Hydrogen Peroxide | Haloacetonitriles |
| Ammonia | Dichloroacetronitrile |
| **ORGANIC OXIDATION BYPRODUCTS** | Bromochloroacetonitrile |
| Aldehydes | Dibromoacetonitrile |
| Formaldehyde | Trichloroacetonitrile |
| Acetaldehyde | Haloketones |
| Glyoxal | 1,1-Dichloropropanone |
| Hexanal | 1,1,1-Trichloropropanone |
| Heptanal | Chlorophenols |
| Carboxylic Acids | 2-Chlorophenol |
| Hexanoic Acid | 2,4-Dichlorophenol |
| Heptanoic Acid | 2,4,6-Trichlorophenol |
| Oxalic Acid | Chloropicrin |
| Assimilable Organic Carbon | Chloral Hydrate |
| | Cyanogen Chloride |
| | N-Organochloramines |
| | MX* |

* 3-Chloro-4-(dichloromethyl)-5-hydroxy-2(5H)-furanone

## Table 2-9. Status of Health Information for Disinfectants and DBPs

| Contaminant | Cancer Classification[1] |
|---|---|
| Chloroform | B2 |
| Bromodichloromethane | B2 |
| Dibromochloromethane | C |
| Bromoform | B2 |
| Monochloroacetic Acid | -- |
| Dichloroacetic Acid | B2 |
| Trichloroacetic Acid | C |
| Dichloroacetonitrile | C |
| Bromochloroacetonitrile | -- |
| Dibromoacetonitrile | C |
| Trichloroacetonitrile | -- |
| 1,1-Dichloropropanone | -- |
| 1,1,1-Trichloropropanone | -- |
| 2-Chlorophenol | D |
| 2,4-Dichlorophenol | D |
| 2,4,6-Trichlorophenol | B2 |
| Chloropicrin | -- |
| Chloral Hydrate | C |
| Cyanogen Chloride | -- |
| Formaldehyde | B1[2] |
| Chlorate | -- |
| Chlorite | D |
| Bromate | B2 |
| Ammonia | D |
| Hypochlorous Acid | -- |
| Hypochlorite | -- |
| Monochloramine | -- |
| Chlorine Dioxide | D |

(1) The scheme for categorizing chemicals according to their carcinogenic potential is as follows:*

| | |
|---|---|
| Group A: Human Carcinogen | Sufficient evidence in epidemiologic studies to support casual association between exposure and cancer. |
| Group B: Probable Human Carcinogen | Limited evidence in epidemiologic studies (Group B1) and/or sufficient evidence from animal studies (Group B2) |
| Group C: Possible Human Carcinogen | Limited evidence from animal studies and inadequate or no data in humans |
| Group D: Not Classifiable | Inadequate or no human and animal evidence of carcinogenicity |
| Group E: No Evidence of Carcinogenicity for Humans | No evidence of carcinogenicity in at least two adequate animal tests in different species or in adequate epidemiologic and animal studies. |

* EPA is in the process of revising the Cancer Guidelines
Source: USEPA, 1996

(2) Based on inhalation exposure.

## Table 2-10. Conditions of Formation of DBPs

| By-product | Conditions of Formation | | |
|---|---|---|---|
| | Chlorination at pH 5.0 | Chlorination at pH 7.0 | Chlorination at pH 9.4 |
| Total Trihalomethanes | Lower formation | | Higher formation |
| Trichloroacetic Acid | Similar formation to that at pH 7.0 | Similar formation to that at pH 5.0 | Lower formation |
| Dichloroacetic Acid | Similar formation to that at pH 5.0 and 9.4 - perhaps slightly higher at pH 7.0 | Similar formation to that at pH 5.0 and 9.4- perhaps slightly higher at pH 7.0 | Similar formation to that at pH 5.0 and 7.0 - perhaps slightly higher at pH 7.0 |
| Monochloroacetic Acid | At concentrations <5 μg/L, trends not discernible | At concentrations <5 μg/L, trends not discernible | At concentrations <5 μg/L, trends not discernible |
| Dibromoacetic Acid | At concentrations <1 μg/L, trends not discernible | At concentrations <1 μg/L, trends not discernible | At concentrations <1 μg/L, trends not discernible |
| Chloral Hydrate | Similar formation to that at pH 7.0 | Similar formation to that at pH 5.0 | Forms within 4 hours; decays over time to <5 μg/L |
| Chloropicrin | At concentrations <1 μg/L, trends not discernible | At concentrations <1 μg/L, trends not discernible | At concentrations <1 μg/L, trends not discernible |
| Dichloroacetonitrile | Higher formation | Forms within 4 hours; decays over time to <5 μg/L | Concentrations <2 μg/L, trends not discernible |
| Bromochloroacetonitrile | At concentrations <2 μg/L, trends not discernible | At concentrations <2 μg/L, trends not discernible | At concentrations <2 μg/L, trends not discernible |
| Dibromoacetonitrile | At concentrations <.5 μg/L, trends not discernible | At concentrations <.5 μg/L, trends not discernible | At concentrations <.5 μg/L, trends not discernible |
| Trichloroacetonitrile | Not detected | Not detected | Not detected |
| 1,1,1-Trichloropropanone | Higher formation | At concentrations <2 μg/L, trends not discernible | Not detected |

Source: Stevens et al., 1989.

# DBP Formation and Control

## *DBP Formation*

Halogenated organics are formed when natural organic matter (NOM) reacts with free chlorine or free bromine. Free chlorine is normally introduced into water directly as a primary or secondary disinfectant. Free bromine results from the oxidation by chlorine of the bromide ion in the source water. Factors affecting the formation of these halogenated DBPs include type and concentration of NOM, chlorine form and dose, time, bromide ion concentration, pH, organic nitrogen concentration, and temperature. Organic nitrogen significantly influenced the formation of nitrogen containing DBPs, including haloacetonitriles, halopicrines, and cyanogen halides . The formation of DBPs is strongly related to TOC at the point of disinfection. DBP formation also correlates with the amount of chlorine consumed . Stevens et al. (1989) found that higher TTHM formation occurs at high pH (9.4) than at low pH (5.0) while HAA showed no clear trend as a function of pH. A survey of 35 water utilities conducted by MWDSC . showed the median TTHM and HAA concentrations measured as 39 and 19 µg/L (i.e. more THMs than HAAs are formed). However, a subsequent study by Singer et al. (1995) found a reversal in dominance, with more HAAs than THMs produced in waters from North Carolina utilities. They postulated that the reason for this change is due to the lower pH levels and differences in TOC and bromide concentrations in the North Carolina waters. Pourmoghaddas et al. (1993) showed that brominated and mixed brominated/chlorinated THMs and HAAs are formed when using chlorine in the presence of bromide.

The occurrence of THMs and HAAs is important because regulatory limits are placed on both groups of compounds. One water utility may therefore find that its chlorination practice is limited by the production of THMs, while another will find that HAAs limit the use of chlorine. This distribution of THMs and HAA is a function of the TOC and bromide concentration in the water, as well as the pH during chlorination.

Of note, Chlorate is produced as a byproduct when hypochlorite degrades during storage.

## ***DBP Control***

DBPs can be controlled by several means, including removing the DBP precursors, modifying the chlorination strategy, changing disinfectants, or removing the DBP itself. Because DBPs are difficult to remove once they are formed, control strategies typically focus on the first three

methods. Studies have shown that removal of TTHM precursors tends to remove the formation potential for the other DBPs. Generally, aggregate DBP formation will decrease as the removal of TOC increases. Recent research indicates that moving the point of chlorination back into the treatment process can reduce the formation of DBPs. Summers et al. (1997) recently summarized the results from four studies evaluating the impact of pretreatment on DBP formation. Jar tests were conducted to simulate the water treatment through rapid mix, coagulation, flocculation, and sedimentation. Chlorine was added at various points in the jar testing to simulate the impact of various dose points on production of DBPs. The results clearly demonstrate the benefits of delaying the point of chlorination downstream in the treatment train to take advantage of precursor removal during initial flocculation and sedimentation processes. Table 2-21 summarizes the results from this study.

**Table 2-21. Percent Reduction in DBP Formation by Moving Chlorination Point Later In Treatment Train**

| Chlorination point | TTHM Baseline (%) | TTHM Enhanced (%) | HAA5 Baseline (%) | HAA5 Enhanced (%) |
|---|---|---|---|---|
| Pre rapid mix | Basis | 17 | Basis | 4.7 |
| Post rapid mix | 1.6 | 21 | 5.3 | 21 |
| Mid flocculation | 8.7 | 36 | 14 | 36 |
| Post sedimentation | 21 | 48 | 35 | 61 |

Notes: Source: USEPA, 1997b based on Summers et al., 1997.
Baseline = Baseline coagulant (alum) dose for optimal turbidity removal (~30 mg/L)
Enhanced = Enhanced coagulant (alum) dose for optimal TOC removal (~52 mg/L)

Table 2-21 also shows the benefit of enhanced coagulation to reduce DBP production. The THM reduction of 21 percent by moving the chlorination point to post sedimentation is more than doubled to 48 percent by enhanced coagulation. The HAA removal increases from 45 to 61 percent under enhanced coagulation with post sedimentation chlorination.

Therefore, DBP control by selecting the optimal dose location and conditions along with enhanced precursor removal can significantly reduce DBP formation at low added cost. White (1992) suggested that pretreatment goals should include: 1) maximizing THM precursor removal; 2) reducing ammonia-N concentration to 0.10 mg/L; 3) reducing organic-N concentration to 0.05 mg/L; and 4) limiting 15-minute chlorine demand to 0.5 mg/L. These guidelines should improve raw water quality sufficiently to allow the use of the free chlorine residual process without exceeding the EPA MCLs for TTHMs.

Another usage of Chlorine in water treatment

- Taste and odor control;
- Prevention of algal growths;
- Maintenance of clear filter media;
- Removal of iron and manganese;
- Destruction of hydrogen sulfide;
- Bleaching of certain organic colors;
- Maintenance of distribution system water quality by controlling slime growth;
- Restoration and preservation of pipeline capacity;
- Restoration of well capacity, water main sterilization; and
- Improved coagulation by activated silica.

## 2.8.1 Advantages and Disadvantages of Chlorine Use

The following list presents selected advantages and disadvantages of using chlorine as a disinfection method for drinking water (Masschelein, 1992; Process Applications, Inc., 1992). Because of the wide variation of system size, water quality, and dosages applied, some of these advantages and disadvantages may not apply to a particular system.

## Advantages

- Oxidizes soluble iron, manganese, and sulfides
- Enhances color removal
- Enhances taste and odor
- May enhance coagulation and filtration of particulate contaminants
- Is an effective biocide
- Is the easiest and least expensive disinfection method, regardless of system size
- Is the most widely used disinfection method, and therefore, the best known
- Is available as calcium and sodium hypochlorite. Use of these solutions is more advantageous for smaller systems than chlorine gas because they are easier to use, are safer, and need less equipment compared to chlorine gas
- Provides a residual.

## Disadvantages

- May cause a deterioration in coagulation/filtration of dissolved organic substances
- Forms halogen-substituted byproducts
- Finished water could have taste and odor problems, depending on the water quality and dosage
- Chlorine gas is a hazardous corrosive gas
- Special leak containment and scrubber facilities could be required for chlorine gas
- Typically, sodium and calcium hypochlorite are more expensive than chlorine gas
- Sodium hypochlorite degrades over time and with exposure to light
- Sodium hypochlorite is a corrosive chemical
- Calcium hypochlorite must be stored in a cool, dry place because of its reaction with moisture and heat
- A precipitate may form in a calcium hypochlorite solution because of impurities, therefore, an antiscalant chemical may be needed
- Higher concentrations of hypochlorite solutions are unstable and will produce chlorate as a byproduct
- Is less effective at high pH

- Forms oxygenated byproducts that are biodegradable and which can enhance subsequent biological growth if chlorine residual is not maintained.

Release of constituents bound in the distribution system (e.g., arsenic) by changing the redoxstate.

# Summary Table

**Table 2-22. Summary of Chlorine Disinfection**

| Consideration | Description |
|---|---|
| Generation | Chlorination may be performed using chlorine gas or other chlorinated compounds that may be in liquid or solid form. Chlorine gas can be generated by a number of processes including the electrolysis of alkaline brine or hydrochloric acid, the reaction between sodium chloride and nitric acid, or the oxidation of hydrochloric acid. Since chlorine is a stable compound, chlorine gas, sodium hypochlorite, and calcium hypochlorite are typically produced off-site by a chemical manufacturer. |
| Primary uses | The primary use of chlorination is disinfection. Chlorine also serves as an oxidizing agent for taste and odor control, prevention of algal growths, maintaining clear filter media, removal of iron and manganese, destruction of hydrogen sulfide, color removal, maintaining the water quality at the distribution systems, and improving coagulation. |
| Inactivation efficiency | The general order of increasing chlorine disinfection difficulty is bacteria, viruses, and then protozoa. Chlorine is an extremely effective disinfectant for inactivating bacteria and highly effective viricide. However, chlorine is less effective against *Giardia* cysts. *Cryptosporidium* oocysts are highly resistant to chlorine. |
| Byproduct formation | When added to the water, free chlorine reacts with NOM and bromide to form DBPs, primarily THMs, some haloacetic acids (HAAs), and others. |
| Point of application | Raw water storage, precoagulation/post-raw water storage, presedimentation/ postcoagulation, postsedimentation/prefiltration, post filtration (disinfection), or in the distribution system. |
| Special considerations | Because chlorine is such a strong oxidant and extremely corrosive, special storage and handling considerations should be considered in the planning of a water treatment plant. Additionally, health concerns associated with handling and use of chlorine is an important consideration. |

# Other Uses of Disinfectants in Water Treatment

Disinfectants are used for more than just disinfection in drinking water treatment. While inactivation of pathogenic organisms is a primary function, disinfectants are also used oxidants in drinking water treatment for several other functions:

- Minimization of DBP formation;
- Oxidation of iron and manganese;
- Prevention of regrowth in the distribution system and maintenance of biological stability;
- Removal of taste and odors through chemical oxidation;
- Improvement of coagulation and filtration efficiency;
- Prevention of algal growth in sedimentation basins and filters;
- Removal of color.

## Minimization of DBP Formation

Strong oxidants may play a role in disinfection and DBP control strategies in water treatment.

Several strong oxidants, including potassium permanganate and ozone, may be used to control DBP precursors.

Potassium permanganate can be used to oxidize organic precursors at the head of the treatment plant, thus minimizing the formation of byproducts at the downstream disinfection stage of the plant.

The use of ozone for oxidation of DBP precursors is currently being studied. Early work has shown that the effects of ozonation, prior to chlorination, were highly site-specific and unpredictable. The key variables that seem to determine the effect of ozone are dose, pH, alkalinity, and the nature of the organic material. Ozone has been shown to be effective for DBP precursor reduction at low pHs. However, at higher pHs (i.e., above 7.5), ozone may actually increase the amount of chlorination byproduct precursors.

## Oxidation of Iron and Manganese

Iron and manganese occur frequently in ground waters but are less problematic in surface waters.

Although not harmful to human health at the low concentrations typically found in water, these compounds can cause staining and taste problems. These compounds are readily treated by oxidation to produce a precipitant that is removed in subsequent sedimentation and filtration processes. Almost all the common oxidants except chloramines will convert ferrous (2+) iron to the ferric (3+) state and manganese (2+) to the (4+) state, which will precipitate as ferric hydroxide and manganese dioxide, respectively. The precise chemical composition of the precipitate will depend on the nature of the water, temperature, and pH.

Table  shows that oxidant doses for iron and manganese control are relatively low. In addition, the reactions are relatively rapid, on the order of seconds while DBP formation occurs over hours. Therefore, with proper dosing, residual chlorine during iron and manganese oxidation is therefore relatively low and short lived. These factors reduce the potential for DBP formation as a result of oxidation for iron and manganese removal.

**Table 2-7. Oxidant Doses Required for Oxidation of Iron and Manganese**

| Oxidant | Iron (II) (mg/mg Fe) | Manganese (II) (mg/mg Mn) |
|---|---|---|
| Chlorine, $Cl_2$ | 0.62 | 0.77 |
| Chlorine dioxide, $ClO_2$ | 1.21 | 2.45 |
| Ozone, $O_3$ | 0.43 | 0.88* |
| Oxygen, $O_2$ | 0.14 | 0.29 |
| Potassium permanganate, $KMnO_4$ | 0.94 | 1.92 |

Source: Culp/Wesner/Culp, 1986; Langlais et al., 1991.

* Optimum pH for manganese oxidation using ozone is 8-8.5 Source: Reckhow et al., 1991.

## Prevention of Regrowth in the Distribution System and Maintenance of Biological Stability

Biodegradable organic compounds and ammonia in treated water can cause microbial growth in the distribution system. “Biological stability” refers to a condition wherein the treated water quality does not enhance biological growth in the distribution system. Biological stability can be accomplished in several ways:

- Removing nutrients from the water prior to distribution;
- Maintaining a disinfectant residual in the treated water; and
- Combining nutrient removal and disinfectant residual maintenance.

To maintain biological stability in the distribution system, the Total Coliform Rule (TCR) requires that treated water have a residual disinfectant of 0.2 mg/L when entering the distribution system. A measurable disinfectant residual must be maintained in the distribution system, or the utility must show through monitoring that the heterotrophic plate count (HPC) remains less than 500/100 mL. A system remains in compliance as long as 95 percent of samples meet these criteria. Chlorine, monochloramine, and chlorine dioxide are typically used to maintain a disinfectant residual in the distribution system. Filtration can also be used to enhance biological stability by reducing the nutrients in the treated water. The level of secondary disinfectant residual maintained is low, typically in the range of 0.1-0.3 mg/L, depending on the distribution system and water quality. However, because the contact times in the system are quite long, it is possible to generate significant amounts of DBPs in the distribution system, even at low disinfectant doses. Distribution system problems associated with the use of combined chlorine residual (chloramines), or no residual, have been documented in several instances. The use of combined chlorine is characterized by an initial satisfactory phase in which chloramine residuals are easily maintained throughout the system and bacterial counts are very low. However, problems may develop over a period of years including increased bacterial counts, reduced combined chlorine residual, increased taste and odor complaints, and reduced transmission main carrying capacity. Conversion of the system to free-chlorine residual produces an initial increase in consumer complaints of taste and odors resulting from oxidation of accumulated organic material. Also, it is difficult to maintain a free-chlorine concentration at the ends of the distribution system

## Removal of Taste and Odors Through Chemical Oxidation

Tastes and odors in drinking water are caused by several sources, including microorganisms, decaying vegetation, hydrogen sulfide, and specific compounds of municipal, industrial, or agricultural origin. Disinfectants themselves can also create taste and odor problems. In addition to a specific taste-and odor-causing compound, the

sensory impact is often accentuated by a combination of compounds. More recently, significant attention has been given to tastes and odors from specific compounds such as geosmin, 2-methylisoborneol (MIB), and chlorinated inorganic and organic compounds. Oxidation is commonly used to remove taste and odor causing compounds. Because many of these compounds are very resistant to oxidation, advanced oxidation processes (ozone/hydrogen peroxide, ozone/UV, etc.) and ozone by itself are often used to address taste and odor problems. The effectiveness of various chemicals to control taste and odors can be site-specific. Suffet et al. (1986) found that ozone is generally the most effective oxidant for use in taste and odor treatment. They found ozone doses of 2.5 to 2.7 mg/L and 10 minutes of contact time (residual 0.2 mg/L) significantly reduce levels of taste and odors. Lalezary et al. (1986) used chlorine, chlorine dioxide, ozone, and permanganate to treat earthy-musty smelling compounds. In that study, chlorine dioxide was found most effective, although none of the oxidants were able to remove geosmin and MIB by more than 40 to 60 percent. Potassium permanganate has been used in doses of 0.25 to 20 mg/L. Studies at the Metropolitan Water District of Southern California demonstrated the effectiveness of peroxone (ozone plus hydrogen peroxide) to remove geosmin and MIB in water treatment (Ferguson et al., 1990; Ferguson et al., 1991; Huck et al., 1995). Prior experiences with taste and odor treatment indicate that oxidant doses are dependent on the source of the water and causative compounds. In general, small doses can be effective for many taste and odor compounds, but some of the difficult-to-treat compounds require strong oxidants such as ozone and/or advanced oxidation processes or alternative technologies such as granular activated carbon (GAC) adsorption.

## Improvement of Coagulation and Filtration Efficiency

Oxidants, specifically ozone, have been reported to improve coagulation and filtration efficiency. Others, however, have found no improvement in effluent turbidity from oxidation collected data from a large treatment plant showing that preozonation was more effective than prechlorination to reduce filter effluent turbidities. The cause of the improved coagulation is not clear, but several possibilities have been offered including:

- Oxidation of organics into more polar forms;

- Oxidation of metal ions to yield insoluble complexes such as ferric iron complexes; and
- Change in the structure and size of suspended particles.

## Prevention of Algal Growth in Sedimentation Basins and Filters

Prechlorination is often used to minimize operational problems associated with biological growth in water treatment plants . Prechlorination will prevent slime formation on filters, pipes, and tanks, and reduce potential taste and odor problems associated with such slimes. Many sedimentation and filtration facilities operate with a small chlorine residual to prevent growth of algae and bacteria in the launders and on the filter surfaces. This practice has increased in recent years as utilities take advantage of additional contact time in the treatment units to meet disinfection requirements under the SWTR.

## Removal of Color

Free chlorine is used for color removal. A low pH is favored. Color is caused by humic compounds, which have a high potential for DBP formation. The chlorine dosage and kinetics for color removal are best determined through bench studies.

# الملحق الثانى

# Annex 2

# Water Chlorination
# Principles and Practices

## AWWA MANUAL M20

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# Foreword

This handbook was prepared to accompany the AWWA Workshop on Chlorination. The latter is part of a comprehensive operators' training program developed by an AWWA *ad hoc* committee and the AWWA Office of Education. Two years of intensive development efforts have been expended by some of the country's foremost chlorination experts to produce this course and handbook.

The objectives of the workshop are to acquaint the experienced operator with the theory, application, use, and safety factors pertaining to chlorine; to help prepare the operator for examination for certification; to advance the operator's skill and knowledge in newer, more modern, and practical methods of chlorination; and to advance AWWA water-quality goals.

The AWWA Office of Education expresses its sincere appreciation to all persons and companies who contributed to the preparation, review, and final editing of this handbook. In particular, it wishes to acknowledge the work performed by the authors, Robert Baker, Leo Carroll, and Edmund Laubusch. Special acknowledgment should also be given to those members, past and present, of the *Ad Hoc* Committee for the Development of a Chlorination Workshop, who, by their dedication, interest, and personal involvement, have made the workshop and this handbook possible. Members of the committee include:

Walter Peters, Director of Education, AWWA

R. J. Baker
R. H. Barg
L. J. Carroll
H. A. Faber
M. E. Flentje
H. E. Hudson
R. E. Kjellman
W. S. Kowalski
E. J. Laubusch
D. W. Loiselle
C. V. Roberts



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CHAPTER 1

# Chlorination Development

## Discovery and Early Concepts

Chlorine was "discovered" in an obscure laboratory in Sweden more than 200 years ago (1744). Some 65 years later (1810) it was identified as a chemical element and was named from the Greek word *chloros* (pale green), because of its characteristic color. It was not, however, until several decades later that its use as a disinfectant was recognized.

Until the germ theory of disease was established (around the mid-1880s), odors were thought to be responsible for transmitting diseases, and so it was commonly believed that control of odors would limit the spread of infection. There is no question that the earliest applications of chlorine, beginning about 1835, were aimed at controlling foul odors even though, because of the unsophisticated techniques used, the chlorinous odor imparted to the water being treated was sometimes just as, or more, objectionable than the odor it was meant to overcome. It was not until well into the 1890s that chlorine and chlorine-containing products were evaluated and demonstrated to be effective disinfectants.

## Origin of Water Disinfection

The disinfection of water probably has been practiced for millenniums — although clearly, at first, with no understanding of the principles involved. Historical records show that the boiling of water had been recommended at least as early as 500 B.C.

The earliest recorded use of chlorine directly for water disinfection was on an experimental basis, in connection with filtration studies at Louisville, Ky., in 1896. It was then employed for a short period in 1897 in England, again on an experimental basis, to sterilize water-distribution mains following a typhoid



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epidemic. Its first *continuous* use was in Belgium, beginning in 1902, for the dual objective of aiding coagulation and making water biologically "safe." In North America, the first continuous, municipal application of chlorine to water was in 1908 — to disinfect the 40-mgd Boonton reservoir supply of the Jersey City, N.J., water utility. Contested in the courts, the practice was declared to represent a public health safeguard, and this action paved the way for its rapid extension to other public water supplies throughout North America and elsewhere.

## Rationale of Water Disinfection

Water disinfection, as now ordinarily considered, involves *specialized* treatment for the destruction of harmful, and of otherwise objectionable, "nuisance" organisms. Classically, disinfection processes have been employed to destroy or inactivate disease-producing (pathogenic) organisms — more particularly, bacteria of intestinal origin. Such organisms can survive for weeks at temperatures near 70F, or possibly even for months at lower temperatures. In addition to temperature influences, their survival in water depends on environmental, physiological, and morphological factors, such as pH, oxygen and nutrient supply, dilution, competition with other organisms, resistance to toxic influences, and ability to form spores. Whether these organisms actually will cause disease in man, upon ingestion, depends on their virulence and concentration and also on the individual's vulnerability or susceptibility to them.

Water disinfection obviously also encompasses destruction of disease-producing organisms other than intestinal bacteria, but it does not necessarily imply complete destruction of *all* living organisms, i.e., *sterilization*. (Water disinfection processes seldom have been carried to the point of sterilization, the latter being largely confined to medical practice.) Among the other disease-producing organisms important with regard to water disinfection are a variety of viruses, intestinal protozoa, and some macroorganisms. Additionally, many nuisance organisms of esthetic or economic significance, both plant and animal, are sometimes vulnerable to disinfection and can be partially or completely controlled by suitable treatment.

## Means of Disinfection

Water disinfection can be accomplished in several ways. Excluding those water-treatment processes (such as sedimentation, coagulation, filtration, etc.) that may result in the partial removal of potentially infective or objectionable organisms, the more specific processes of current significance include one or a combination of the following: (1) physical treatment, such as through storage or by the application of heat or other physical agents; (2) irradiation, such as by ultraviolet; (3) metal ions, such as copper or silver; (4) alkalis and acids; (5) surface-active chemicals, such as quaternary ammonium compounds; and (6) oxidants, such as halogens, ozone, and other inorganic or organic materials.

Except for chlorine and some of its compounds (and heat, in the case of individual water-supply systems), most other disinfectants have one or more serious limitations that preclude their general acceptance and adaptability, by themselves, for municipal water-treatment operations.

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## Evolution of Chlorination Materials

During the infancy of water chlorination, the only commercial sources of chlorine were dry chlorine-containing *compounds*, such as chlorinated lime (also termed chloride of lime, bleaching powder, etc.) and sodium hypochlorite bleach solutions. The poor stability and variable effective chlorine content of the compounds then available caused many operating difficulties, and often inadequate disinfection dosages were used. Moreover, the comparatively crude equipment feeders yielded erratic results.

In 1909, *liquid chlorine* — the element in compressed form — became available commercially, and in the following year it was experimentally employed for water disinfection at Fort Myer, Va. The chlorine was added to the water by using a simple dry-gas feeder. In 1912, the first full-scale successful use of liquid chlorine was undertaken to control a recurring outbreak of typhoid at Niagara Falls, N.Y.; in this case, a solution feeder was employed. Two years later, improved equipment (developed by C. F. Wallace and M. F. Tiernan) to measure chlorine gas, dissolve it in water, and apply the solution was installed at Boonton, N.J., replacing the earlier use of sodium hypochlorite bleach there. These developments paved the way for the future extension of water disinfection techniques utilizing liquid chlorine.

Hypochlorite water chlorination gradually decreased in popularity, but it received renewed stimulus with the commercial availability, in 1928, of high-test calcium hypochlorite, a more stable and active material than the various bleaching powders and solutions previously available. Today, the principal sources of chlorine for water disinfection and other chlorination treatment are: (1) in elemental form, as a liquefied compressed gas (in commerce); (2) as "high-test calcium hypochlorite"; (3) as chlorine bleach solutions (not to be confused with elemental liquid chlorine or with water solutions of chlorine gas); and (4) to a much lesser extent as chlorine dioxide.

## Evolution of Chlorination Practices

In its initial applications, chlorination was a final adjunct to filtration and other existing water-purification processes or, in some instances, it was the only treatment provided. Very soon, however, advocates of chlorination began to develop more sophisticated chlorination practices.

Beginning about 1915, the influence of ammonia on the disinfecting capacity of chlorine was observed and reported by many investigators. The widespread adoption of *chlorine-ammonia treatment* followed recognition that the combination of chlorine and ammonia produced a more stable disinfecting residual than that produced by chlorine alone, and that the process could be applied to limit the development of objectionable tastes. Chlorine-ammonia treatment later declined in popularity, however, largely due to the advent and development of "free residual" chlorination processes and to realization of the superior bactericidal efficiency of hypochlorous acid. (Currently, the principal application is for posttreatment, to provide long-lasting chloramine residuals in potable-water-distribution systems.)

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The *"breakpoint"* concept of water chlorination began to emerge in 1939 when it was reported that some waters exhibit a break in the chlorine residual curve upon addition of a supplementary amount of chlorine. Impetus for its adoption followed when it was demonstrated that the instantaneous development of full color in the orthotolidine colorometric test signified the presence of the more desirable, free chlorine residual. Various test prodecures for distinguishing between free and combined residuals were subsequently introduced and perfected (mostly during the 1940s and later).

As early as about 1900, the use of *chlorine dioxide* for water disinfection had been tried experimentally, but practical application proved unfeasible because of the instability of the gaseous material, and because of the necessity for on-the-spot generation. At about the time breakpoint chlorination was introduced, sodium chlorite was commercially developed as a bleaching agent. A dry flaked or powdered material that is readily soluble in water, it readily reacts with chlorine- to produce an aqueous solution of chlorine dioxide. The on-site generation method was experimentally employed, and was finally adopted on a full-scale basis, in 1944 at Niagara Falls, N. Y., for controlling phenolic tastes and odors.

Although the bactericidal properties of chlorine dioxide are about equal to those of chlorine (when optimally applied), it rarely is used solely for the purpose of disinfection. The literature contains several reports, however, of its successful application for oxidation of iron, manganese, phenolic and chlorophenolic compounds, for control of certain algae, and for control of taste and odor problems in general.

## Evolution of Chlorination Control Practices

Improvements in chlorine disinfection materials and in chemical feeders substantially contributed to the popularity and widespread adoption of water chlorination. Disinfection dosages originally were based largely on the application of fixed amounts of chlorine. Soon it became apparent that no provisions were being made for the effects of variations in water quality and of fluctuation in chlorine demand. Gradually, the concept of varying chlorine dosage on the basis of residual chlorine was established, and iodometric methods for qualitative and quantitative assay of residual chlorine were developed. The use of orthotolidine as a qualitative indicator of residual chlorine was proposed in 1909, and later was extended quantitatively by the development of calorimetric standards for its use. Chlorine disinfection finally was established on a scientific basis (in the period from 1917 to 1919) when the suitability and reliability of the orthotolidine test for even the smallest water supplies were demonstrated. Since then, a better understanding of water chlorination processes has brought many refinements in that test and in the development of other tests, notably: the orthotolidine-arsenite calorimetric test to distinguish forms of residual chlorine; amperometric differential titration; various other differential chemical tests for available chlorine; and test methods applicable to chlorine dioxide.

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## Current Status of Chlorination

The past half century of experiment, development, and refinement has crystallized certain practices and control procedures that ensure effectiveness of chlorine for disinfection and other treatment applications. No other water-treatment process enjoys such popularity and almost universal adoption, particularly in the United States.

In 1962, the latest year for which published data are available, more than 10,000 US water-treatment facilities, serving about 135 million persons, employed chlorine disinfection. (Although liquid chlorine surpasses hypochlorites for these purposes, particularly at larger facilities, hypochlorite chlorine forms are by no means obsolete or diminishing in importance. During the period 1948-1962, the number of US facilities using liquid chlorine increased by about 64 per cent, whereas the number using hypochlorites increased by nearly 150 per cent. This disparity undoubtedly is due to the adoption of chlorine disinfection at small treatment plants where hypochlorites may be especially appropriate.)

Gaseous chlorine is the least expensive form of chlorine that can be employed where material requirements are substantial. In small water-utility installations, and for field, emergency, or other specialized uses, some chlorine-containing materials might be more satisfactory and economical. Among such materials are various hypochlorites, chlorinated lime, chlorine dioxide, and chloramines produced from the purposeful combination of chlorine with ammonia or other nitrogenous material.

Within the past decade or two, the functional advantage of treatment employing amounts of chlorine greater than those formerly considered adequate has become clearly appreciated. The wider adoption of continuous chlorine treatment can be expected to continue to develop inasmuch as high-rate chlorination provides a means of achieving high standards of bacteriological quality and of improving operation of other water-treatment processes. In addition to its continuous application as an integral part of potable water treatment, a number of other specialized applications of chlorine have been proven, such as the disinfection of new or repaired mains, of tanks and reservoirs, and of wells. (Practically all states require that all new or repaired sections of potable-water-distribution systems be suitably chlorinated before being put into service.)

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CHAPTER 2

# Chlorination Materials

## Elemental Chlorine

**Physical properties.** In the gaseous state, chlorine is greenish-yellow in color and is about 2 1/2 times as heavy as air. In commerce, it is available as a compressed liquid that is amber colored and about 1 1/2 times as heavy as water. If liquid chlorine is unconfined, it rapidly vaporizes to gas (one volume of liquid yields about 450 volumes of gas).

Since chlorine confined in a container may exist as a gas, as a liquid, or as both, any consideration of liquid chlorine includes that of gaseous chlorine. As will be noted later, gas vapor pressure of chlorine in a container is a function of temperature and is independent of the contained volume of chlorine. Thus, gage pressure of a container having 1 lb of chlorine will be essentially the same as if it contained 100 lbs of chlorine (under the same temperature conditions). The temperature-pressure characteristics of chlorine are shown in Fig. 1. Note that when the chlorine temperature increases so does the pressure.

The volume-temperature characteristics of chlorine in a container loaded to the limit authorized by the US Dept. of Transportation are shown in Fig. 2. Note that a shipping container filled to its authorized limit becomes completely full of liquid at approximately 154F. Temperatures beyond that point may produce pressures that could result in hydrostatic rupture of the container. Safety devices (described in Chap. 7) always are provided to relieve excessive pressures accompanying dangerous temperature increases.

Chlorine is only slightly soluble in water; its maximum solubility is approximately 1 per cent at 49.2C. At temperatures below this point, it combines with water to form chlorine "ice" — a crystalline substance. (When the water supplying a chlorinator is at a sufficiently low temperature to cool moist chlorine gas to the point at which chlorine ice is formed, accumulations on the needle valve and gas outlet tube develop and erratic feed results; this condition can be corrected by slightly warming either the portion of the water that comes in contact with the chlorine or the gas feed line.) Because the vapor pressure of chlorine increases with rising temperature, its solubility also decreases. At 212F



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chlorine is insoluble in water; between 49.2F and 212F, chlorine dissolved in water forms a weak, corrosive mixture of hydrochloric and hypochlorous acids. The corrosivity of chlorine-water solutions creates problems in handling.

**Chemical properties.** Under specific conditions chlorine reacts with most elements, sometimes extremely rapidly. Because of its great affinity for hydrogen, chlorine removes hydrogen from some of its compounds, such as in the reaction with hydrogen sulfide. It also reacts, as later discussed, with ammonia or other nitrogen-containing compounds to form various mixtures of chloramines. It reacts much the same with organic materials, sometimes with explosive violence.

Although it is neither explosive nor flammable, chlorine (like oxygen) is capable of supporting combustion of certain substances. It should, therefore, be handled and stored away from other compressed gases (e.g., ammonia) and flammable materials.

Most common metals are not affected at normal temperatures by *dry* chlorine, either gas or liquid. Chlorine is, however, reactive with aluminum and ignites carbon steel at temperatures above about 450F. Moist chlorine is

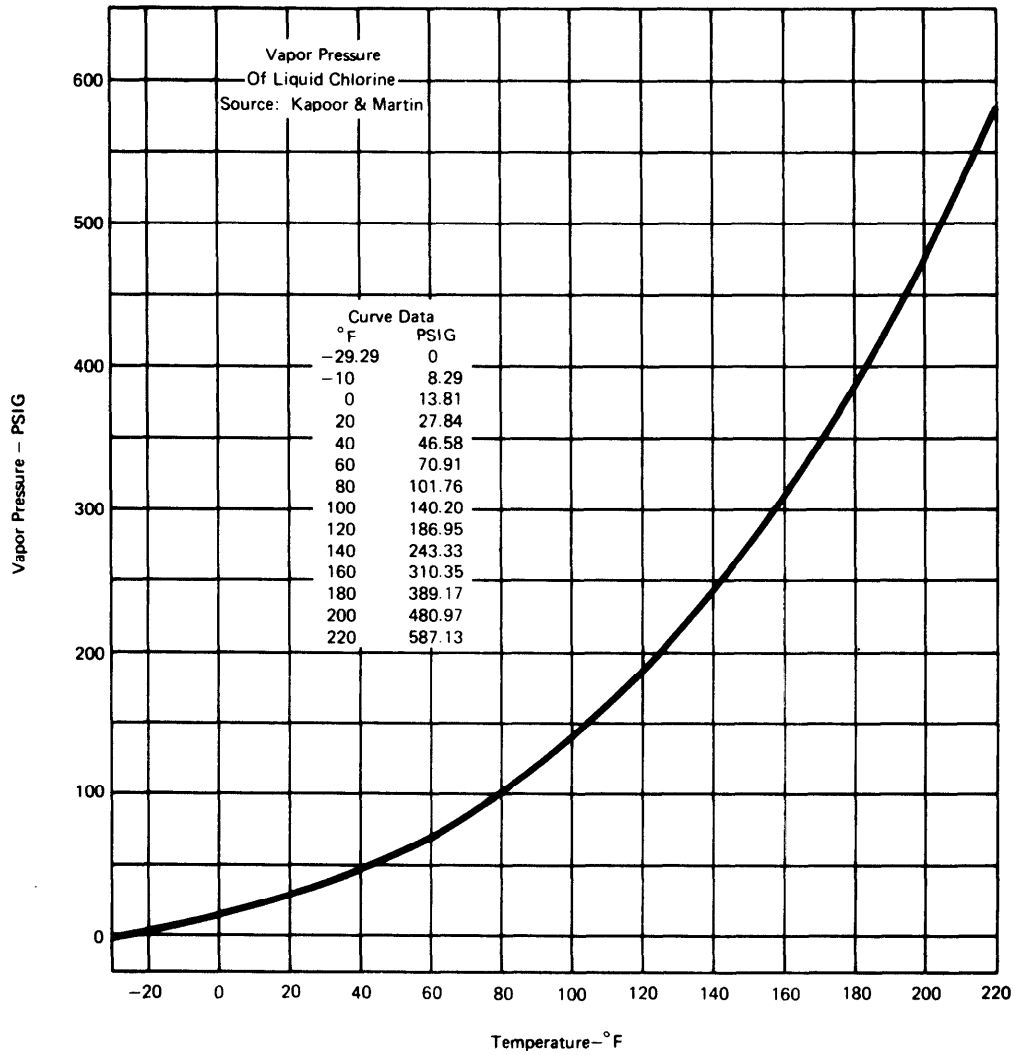


**Fig. 1.** Vapor Pressure of Liquid Chlorine

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corrosive to all common metals with the exception of gold, silver, platinum, titanium, and certain specialized alloys.

**Physiological effects.** Chlorine gas is primarily a respiratory irritant, and concentrations in air above 3 ppm by volume can usually be detected (by smell) by most persons. Chlorine causes varying degrees of irritation of the skin (particularly in the presence of moisture), mucus membranes, and the respiratory system, depending on the concentration and duration of exposure. In severe cases death can occur from suffocation. However, the extremely irritating effect of the gas makes it unlikely that any person would deliberately remain in a dangerous atmosphere unless he were unconscious or trapped.

Liquid chlorine may cause skin and eye bums upon contact with these tissues. (When unconfined in a container, chlorine vaporizes rapidly, producing the same effects as the gas.)

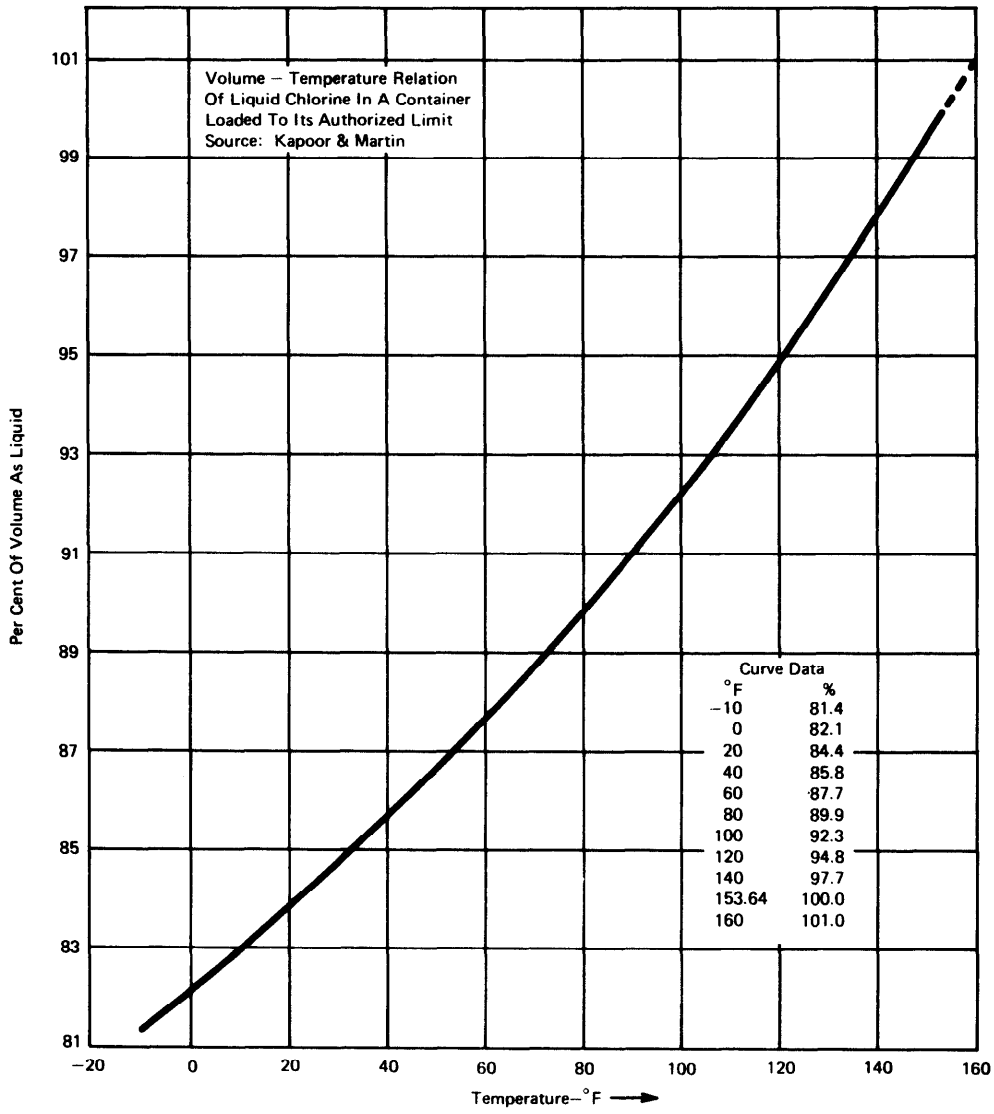


**Fig. 2.** Volume-Temperature Relation of Liquid Chlorine in a Container Loaded to Its Authorized Limit

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Chlorine produces no known cumulative effects and complete recovery usually can be expected to occur shortly following mild, short-term acute exposures. An 8-hr, time-weighted exposure of up to 1 ppm (approximately 3 mg/cu m) is the current national consensus standard provided for in the 1971 Occupational Safety and Health Act. Concentrations of 40-60 ppm for 30-60 min are dangerous, and higher concentrations, even for very brief periods, can be fatal.

## Hypachlorites

Hypochlorites are salts of hypochlorous acid; they can be classified as either *dry* or *liquid* depending on commercial availability. The same equilibria are established in water as with gaseous chlorine. The significant difference is in the effect of pH on the relative amounts of $OCl^-$ and HOC1 at equilibrium (Eqs Z-3 to Z-5). Solutions of hypochlorite contain an excess of alkali, which tends to increase the pH.

Calcium hypochlorite is the predominant dry form in use in North America. Lithium hypochlorite, particularly for use in the swimming pool market, is a more recent competitive product. Sodium hypochlorite is the only liquid hypochlorite disinfectant in current use. There are several grades and proprietary forms available.

Pound for pound of available chlorine, hypochlorite chlorine compounds have oxidizing power equivalent to gaseous chlorine and can be employed for the same purposes in water treatment. The choice of chlorine form depends to a large extent on material requirements, safety considerations, and economics of use. Hypochlorite chlorine compounds are especially suitable for emergency and field disinfection and at smaller installations, and results comparable to those of gaseous chlorine can usually be obtained. However, chlorine in gaseous form appears to be the preferred, least costly, and most widely used form for disinfection in the majority of instances. Although rules of thumb cannot be practically applied, experience suggests that hypochlorites are especially adaptable to utilities serving up to about 5,000 persons or where safety considerations are paramount.

**Properties of calcium hypochhorite.** Calcium hypochlorite materials used in water treatment are chemically different from those materials variously marketed for many years as bleaching powder, chloride of lime, or chlorinated lime; the latter are not calcium hypochlorite, $Ca(OC1)_2$, but consist of a relatively unstable mixture of compounds, largely calcium oxychloride. Materials now in common use are so-called “high-test calcium hypochlorites,” containing about 70 per cent available chlorine and marketed under several trade or proprietary names.

High-test calcium hypochlorites are white, essentially nonhygroscopic, corrosive solids that give off a fairly strong, chlorinous odor. Granular, powdered, or tablet hypochlorite forms are commercially available, and all are readily soluble in water.

**Properties of sodium hypochlorite.** Sodium hypochlorite, NaOCl, often referred to as “liquid bleach,” is commercially available only in liquid form —

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usually in concentrations of between 5 and 15 per cent available chlorine. These solutions are clear, light yellow, strongly alkaline, and corrosive, and they also have a strong, chlorinous odor.

**Hypchlorite stability.** High-test hypochlorites, though highly active oxidizers, are relatively stable throughout production, packaging, distribution, and storage. Storage at 86F for about a year might reduce the available chlorine content by about 10 per cent. Cooler storage reduces the deterioration.

All sodium hypochlorite solutions are unstable to some degree and deteriorate more rapidly than dry hypochlorites. Most producers recommend a maximum shelf life of 60-90 days. Because light and heat accelerate decomposition of sodium hypochlorite solutions, product degradation is less pronounced when containers are stored in a dry, cool, and darkened area, or in containers protected from light.

## Chlorine Dioxide

Chlorine dioxide, $C1O_2$, has never been used extensively for municipal water disinfection. It is a powerful germicide (with capability about equal to that of hypochlorous acid) against all known pathogenic organisms. Its potency is unaffected by pH in the usual ranges encountered or by the presence of ammonia. It can be generated and dosed readily, it can be measured without great difficulty, and it is reasonably persistent.

In most circumstances, chlorine dioxide is regarded as only marginally superior to chlorine when well-controlled, free residual chlorination is practiced. Moreover, the necessary on-site generation of aqueous $C1O_2$ from the relatively expensive sodium chlorite, $NaC1O_2$, means that, at equivalent dosages, it inevitably costs more than chlorine:

$$2N_aCIO_2 + C1_2 \longrightarrow 2C1O_2 + 2N_aC1 \quad (\text{Eq } Z-1)$$

For the near future, at least, it appears that chlorine dioxide will be of interest in water disinfection only in situations where a combination of characteristics, such as several parts per million of ammonia-nitrogen plus considerable phenolic material, makes production of high-quality water by chlorine alone especially difficult.

Although chlorine dioxide sometimes is applied alone, it usually is used in conjunction with, and precedes or follows application of, chlorine. If used to destroy chlorophenols, it is applied as posttreatment, in conjunction with postchlorination. If it is used to oxidize iron and/or manganese, it usually is applied prior to coagulation, settling, and filtration, with chlorine subsequently being added for disinfection.

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CHAPTER 3

# Water Chlorination Principles

## Treatment Objectives

Chlorine is used in water treatment (1) for disinfection — to destroy or "inactivate" microorganisms of sanitary significance, (2) for oxidation — to purposefully modify the chemical character of the water to which it is applied, or (3) for both disinfection and oxidation (see Fig. 3). Actually, the toxic character

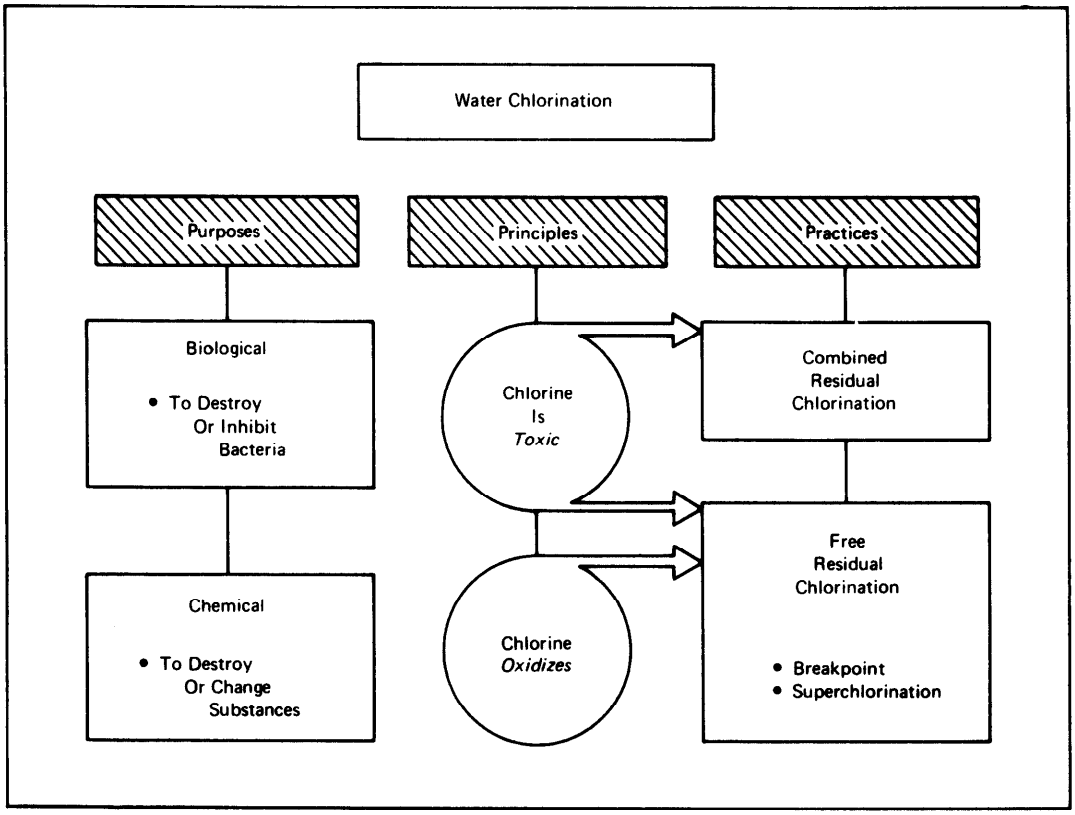


**Fig. 3.** Application of Chlorine in Water Treatment



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of chlorine and some of its compounds is due in large measure to their oxidizing capacity, but oxidizing capacity is not necessarily a measure of germicidal efficiency.

Disinfection is the chief and most common objective of chlorination. In fact, the terms disinfection and chlorination largely have come to imply the same treatment. As discussed later, not all chlorine forms, or their reaction products

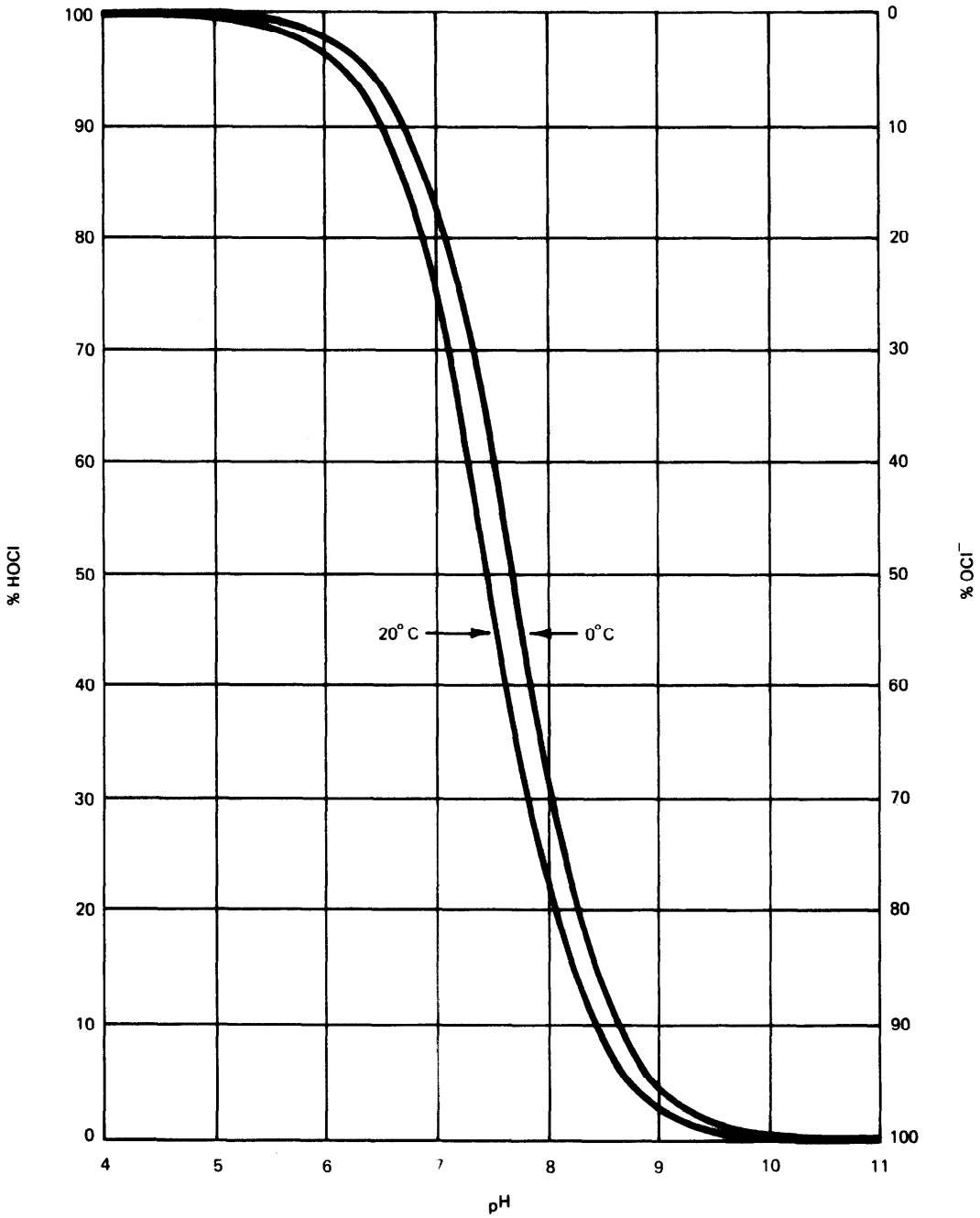


**Fig. 4.** Distribution of HOC1 and $OC1^{-}$ in Water at Indicated pH Levels

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with water, are equally effective or applicable for disinfection and/or oxidation reactions.

Water chlorination has evolved from a relatively crude process application to a highly efficient one, due largely to improved methods of testing the effectiveness of treatment and, through these, to improved definition of the biological and chemical processes involved.

## Reactions With Water

When chlorine is added to chemically pure water, a mixture of hypochlorous (HOC1) and hydrochloric (HC1) acids is formed:

$$Cl_2 + H_2O \rightleftharpoons HOCl + H^+ + Cl^- \qquad \text{(Eq Z–2)}$$

At ordinary water temperatures this reaction is essentially complete within a few seconds. In dilute solution and at pH levels above about 4, the equilibrium shown in Eq 2-2 is displaced to the right and very little $C1_2$ exists in solution. In normal practice (except for the concentrate from solution-feed chlorinators), the amount of chlorine supplied to water does not produce a concentrated solution of such strength as to yield such a low pH. The oxidizing property of the chlorine is, however, retained in the HOC1 formed, and it is also in this form that the principal disinfecting action of aqueous chlorine solutions occurs.

Hypochlorous acid ionizes or dissociates, a practically instantaneous reaction, into hydrogen and hypochlorite ions (note that this reaction also is reversible), the degree of dissociation depending on pH and temperature:

$$HOCl \rightleftharpoons H^+ + OCl^- \qquad \text{(Eq Z–3)}$$

Hypochlorous acid is weak and dissociates poorly at levels of pH below about 6; thus, chlorine exists predominantly as HOC1 at relatively low pH levels. Between pH 6.0 and 8.5 there occurs a very sharp change from undissociated HOC1 to almost complete dissociation. At 20C above about pH 7.5 and at 0C above about pH 7.8, hypochlorite ions $OC1^-$) predominate, and they exist almost exclusively at levels of pH around 9.5 and above (see Fig. 4). The pH of chlorinated water supplies is normally within the range where chlorine may exist as both hypochlorous acid and hypochlorite ion. *Chlorine existing in water as hypochlorous acid and hypochlorite ions is defined as free available chlorine.*

Chlorine-containing compounds, such as high-test (70 per cent available chlorine) calcium hypochlorite, $Ca(OC1)_2$, and sodium hypochlorite, NaOCl, ionize in water and also yield hypochlorite ions:

$$Ca(OCl)_2 + H_2O \rightleftharpoons Ca^{++} + 2OCl^- + H_2O \qquad \text{(Eq Z–4)}$$

$$NaOCl + H_2O \rightleftharpoons Na^+ + OCl^- + H_2O \qquad \text{(Eq Z–5)}$$

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The hypochlorite ions also establish equilibrium with hydrogen ions, depending on the pH, as shown by Eq 2-3. Thus, the same equilibria are established in water regardless of whether gaseous chlorine or hypochlorite chlorine compounds are employed. The important distinction is the resultant pH and hence the relative amounts of HOC1 and $OC1^-$ existing at equilibrium. Chlorine tends to decrease the initial pH, and hypochlorites tend to increase it.

From Eq Z-2, it is evident that the hypochlorous and hydrochloric acids produced tend to lower pH and reduce alkalinity. Each part of chlorine added will neutralize not less than 0.7 part of alkalinity (as $CaCO_3$), and may neutralize 1.4 parts, depending upon the extent to which HOC1 is ionized and the manner in which chlorine is consumed by substances in the water. Hardness-producing materials, principally the carbonates and bicarbonates of calcium and magnesium, tend to buffer water against significant changes in pH that otherwise would accompany the addition of small amounts of acid or alkaline materials. If sufficient amounts of chlorine are applied to deplete natural alkalinity to such an extent that the pH is substantially decreased, however, pH correction may be indicated as auxiliary treatment. Hypochlorites always contain excess alkali for stability and tend to raise pH levels, but usually by insignificant amounts.

### Chlorine-Ammonia Reactions

The reactions of chlorine with inorganic nitrogenous material, such as ammonia, are of great significance in water chlorination processes, especially disinfection, The kinetics of these reactions have been extensively studied. When chlorine is added to water containing natural or added ammonia (ammonium ion exists in equilibrium with ammonia and hydrogen ions), the ammonia reacts with HOC1 to form various chloramines that, like HOC1, retain the oxidizing power of the chlorine. The reactions between chlorine and ammonia may be represented by

$$NH_3 + HOCl \longrightarrow \underset{\text{(Monochloramine)}}{NH_2Cl} + H_2O \qquad \text{(Eq Z–6)}$$

$$NH_2Cl + HOCl \longrightarrow \underset{\text{(Dichloramine)}}{NHCl_2} + H_2O \qquad \text{(Eq Z–7)}$$

$$NHCl_2 + HOCl \longrightarrow \underset{\text{(Trichloramine or nitrogen trichloride)}}{NCl_3} + H_2O \qquad \text{(Eq Z–8)}$$

The distribution of the reaction products is governed by the rates of formation of monochloramine and dichloramine, which are dependent upon pH, temperature, time, and initial $C1:NH_3$ ratio. In general, high $C1:NH_3$ ratios, low temperatures, and low pH levels favor dichloramine formation. Figure 5 illustrates the relative amounts of monochloramine and dichloramine likely to be formed at various pH levels when the $C1:NH_3$ ratio is equimolar (5:1 on a weight basis). Although reported data differ to some degree, it is evident that some dichloramine can be anticipated at pH levels below 7. Other work further

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suggests that at pH levels below 7.5, some odorous nitrogen trichloride also can be anticipated.

Chlorine also reacts with organic nitrogenous materials, such as proteins and amino acids, to form organic chloramine complexes. *Chlorine existing in water in chemical combination with ammonia-nitrogen or organic nitrogen compounds is defined as combined available chlorine.*

The oxidizing capacity of free chlorine solutions varies with pH because of variations in the resultant HOC1:$OC1^-$ ratios, decreasing at high pH levels where $OC1^-$ predominates. This principle applies also in the case of chloramine solutions as a result of varying $NHC1_2$:$NH_2C1$ ratios, and where monochloramine predominates at high pH levels. Chloramines are less potent disinfectants and are less reactive than free available chlorine forms.

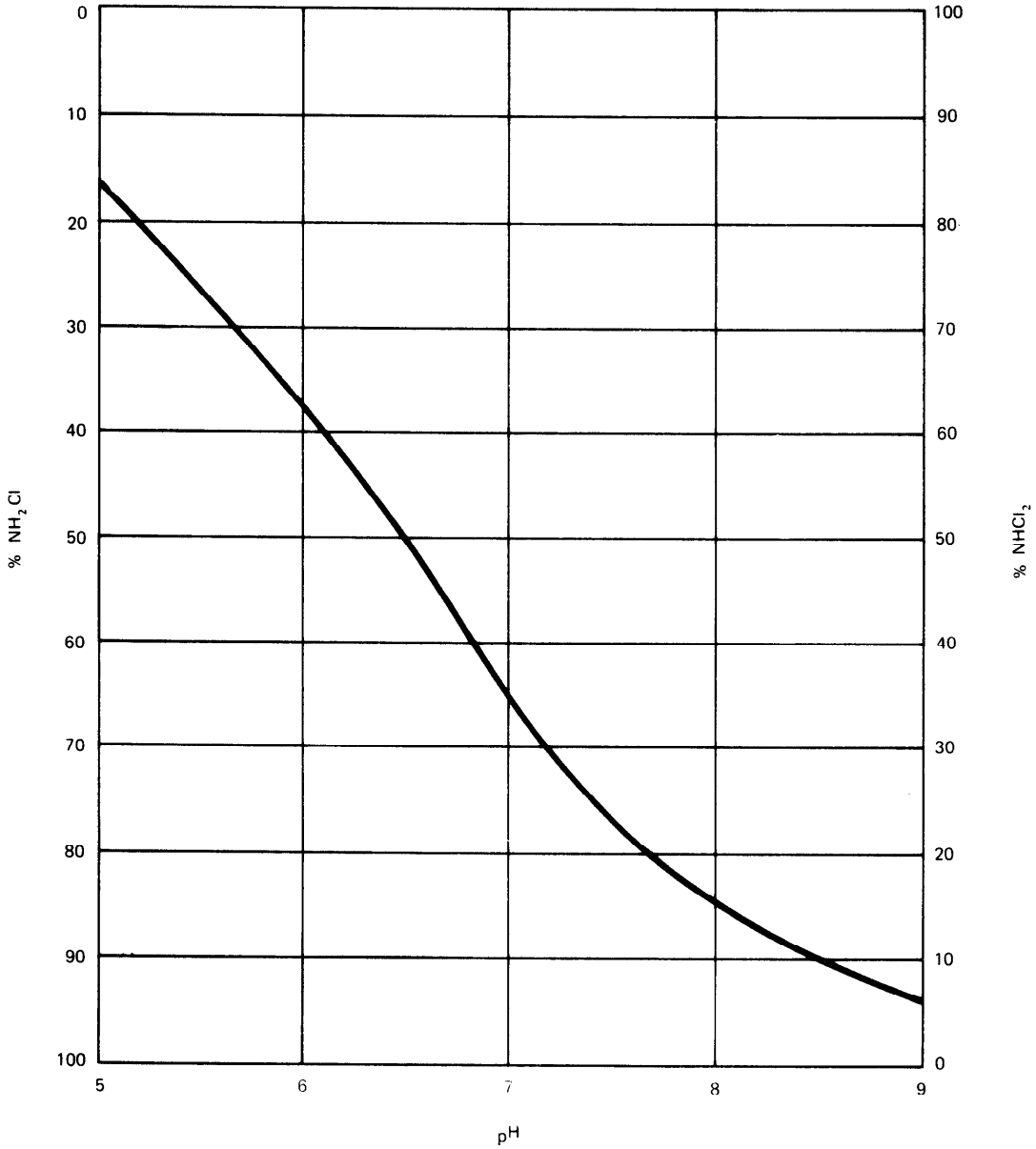


**Fig. 5.** Distribution of Chloramines in 25C Water at Indicated pH Levels and $C1_2$: $NH_3$ (as N) Weight Ratio 5:1

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## Other Chlorine Reactions

The reaction between hydrogen sulfide and chlorine, generally very rapid, is typical of that which occurs with inorganic reducing materials:

$$H_2S + 4Cl_2 + 4H_2O \longrightarrow H_2SO_4 + 8HCl \qquad \text{(Eq. Z–9)}$$

In the reduced form of a chloride ion, $C1^-$, oxidizing power of chlorine does not exist. One part of $H_2S$ theoretically can be oxidized by about 8.5 parts of chlorine, and chlor-oxidation treatment sometimes is practiced with this objective in mind; about 10 parts of alkalinity (as $CaCO_3$) are consumed by the reaction that occurs optimally at about pH 9.

Iron and manganese (Fe and Mn) in water also affect chlorination practices; if the pH is high enough for hydroxide formation and if sufficient chlorine is added, ferrous and manganous forms of the metals will be rapidly oxidized to insoluble hydroxide forms. Chloramines do not enter vigorously into these oxidation reactions. For satisfactory manganous oxidation, free available chlorine, as distinct from combined available chlorine, is required. For iron

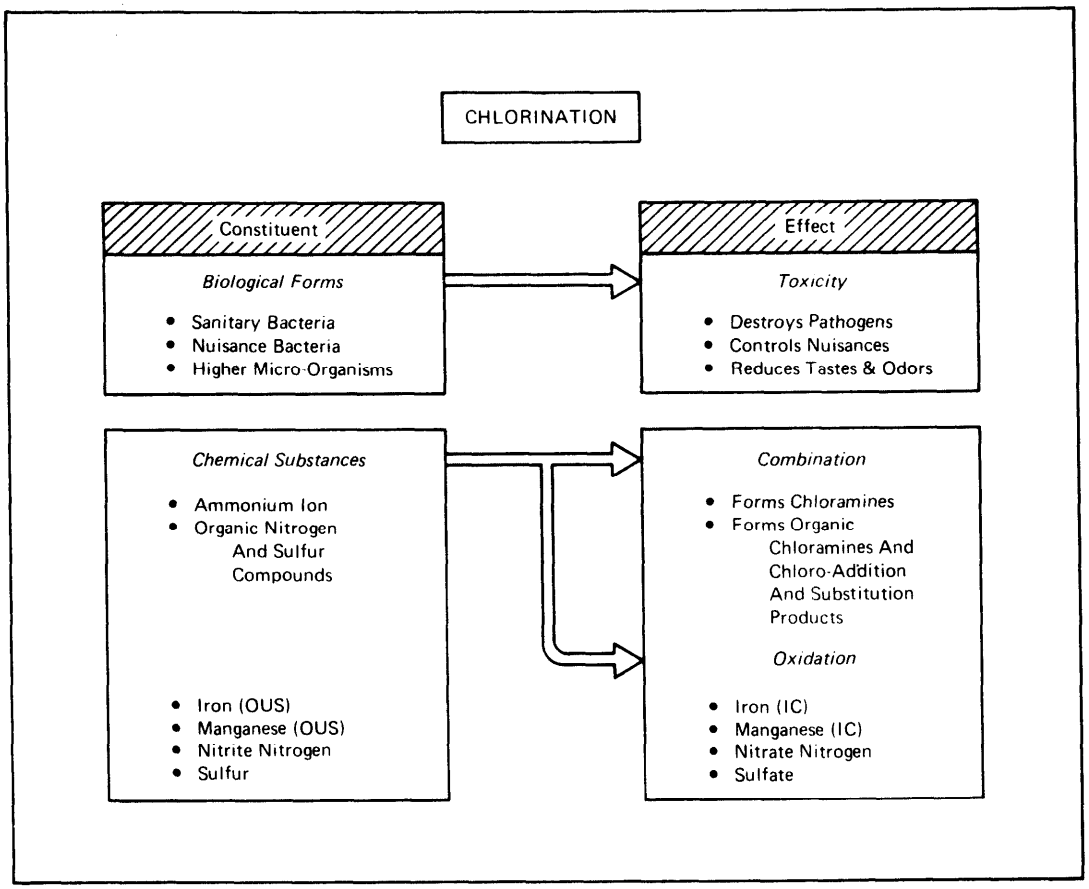


**Fig. 6.** Biological and Chemical Effects of Chlorine in Water

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removal the optimum pH is 7 or above, and for manganese removal the optimum pH is 10. About 0.6 and 1.3 parts of chlorine are required, respectively, for each part of Fe and Mn removed, and about 0.9 and 3.4 parts of alkalinity (as $CaCO_3$) are consumed.

Nitrites are occasionally present in some waters, particularly those contaminated by domestic and certain industrial wastes. They, too, are readily oxidized to nitrates by chlorine, particularly free available chlorine, each part of nitrite consuming about 1.5 parts of chlorine.

A few organic reducing materials may be present in potable waters but their concentration usually is low. However, there are a variety of chlorine-reactive, organic materials in some waters that can exert a substantial influence on chlorine requirements, depending upon available chlorine concentration and reaction time. Some dissolved organic materials react with chlorine rapidly, but, generally, the time required for the completion of most organic reactions is a matter of hours. Under some conditions, chlor-addition or chlor-substitution products might be formed, and under others the organic material may be completely oxidized. (Organic chlorine complexes generally have low bactericidal capacity and are of principal concern in water-chlorination practice because of the chlorine consumed and the tastes and odors they may impart.)

Biological and chemical effects of chlorine in water are illustrated in Fig. 6.

## Available Chlorine

The term "available chlorine content" was established in the early days of chlorination as a basis of comparison of the potential disinfecting power of chlorine compounds with that of elemental chlorine, at a time when it was believed that the gas was 100 per cent utilized or "available" in disinfection reactions. The available chlorine content of a solution is determined by titrating the iodine that the solution will liberate from an acidified iodide solution (see Eq Z-20, p. 41). The calculated weight of elemental chlorine, $C1_2$, required to liberate the same amount of iodine is the "available chlorine content" of the solution. [Because only half of the $C1_2$ molecule is positive in solution (i.e., the chlorine atom of HOC1), the available chlorine content of hypochlorites and chloramines is just twice the "active" (positive) chlorine content. For example, the percentage of chlorine in dichloramine is 82.5, but the available chlorine content is 165.0 per cent.] Thus, the titration described is a quantitative measure of the oxidizing power of a solution, determined by the ability of the solution to release iodine from acidified iodide, and expressed in terms of elemental chlorine. The oxidizing power of materials containing no chlorine also can be expressed in terms of available chlorine. Anhydrous hydrogen peroxide, for example, has an equivalent available chlorine content of 208 per cent. It should be noted, however, that the oxidizing power of a material is not necessarily a measure of its disinfecting efficiency; thus, hydrogen peroxide is a powerful oxidant but a poor water disinfectant.

The per cent available chlorine of various chlorine-containing materials is indicated in Table 1.

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**TABLE 1**
**Per Cent Available Chlorine of Various Chlorine Materials**

| Material | Available Chlorine Per Cent |
|---|---|
| $Cl_2$, chlorine | 100 (by definition) |
| Bleaching powder (chloride of lime, etc.) | 35–37 |
| $Ca(OCl)_2$, calcium hypochlorite | 99.2 |
| Commercial preparations | 70–74 |
| NaOCl, sodium hypochlorite (unstable) | 95.2 |
| Commercial bleach (industrial) | 12–15 |
| Commercial bleach (household) | 3–5 |
| $ClO_2$, chlorine dioxide | 263.0 |
| $NH_2Cl$, monochloramine | 137.9 |
| $NHCl_2$, dichloramine | 165.0 |
| $NCl_3$, nitrogen trichloride | 176.7 |

## Mode of Action

Chlorine disinfection involves a very complex series of events and is influenced by the kind and extent of reactions with chlorine-reactive materials (including nitrogen), temperature, pH, the viability of test organisms, and numerous other factors. Such factors greatly complicate attempts to determine the precise mode of action of chlorine on bacteria and other microorganisms. Over the years several theories have been advanced.

It is generally thought that death of bacterial cells results from a chemical reaction between HOC1 and an enzyme system essential to glucose oxidation and cellular metabolic activity. (The reactions are presumed to involve irreversible HOC1 oxidation of sulfhydryl-containing enzymes; most viruses do not have demonstrable enzymes and yet are inactivated by chlorine.) It also has been suggested that the phenomenon of unbalanced growth can be the effective mechanism of death for microorganisms. Convincing evidence exists that destruction of part of an enzyme system throws the cell out of balance so that, by progress of its own metabolism, the cell dies before its functions can be brought back in balance.

Although the enzyme-deactivation theory does provide an explanation of the extreme sensitivity of certain organisms to chlorine, it does not furnish the complete answer, for enzymes removed from cells are equally susceptible to attack by some oxidants other than chlorine, whereas chlorine is markedly superior to these same oxidants in attacking intact cells. It has been suggested that the difference might be explained on the basis that not only is the ability of a disinfectant to react on a key enzyme system important, but also is its ability to penetrate the cell. The rate of diffusion of the disinfectant into the cell largely determines the rate of disinfection and relative efficiencies of disinfecting agents. Thus, perhaps the superior effectiveness of HOC1 to other chlorine forms is due not to its strong oxidizing power, but rather to its small molecular size and electrical neutrality, which allow it to penetrate the cell readily. Hypochlorite

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ion has little, if any, bactericidal effect, and its negative charge is presumed to impede cell penetration. The greater resistance of spores than of vegetative cells, then, might well be due to the permeability barrier, which is not readily penetrated by such compounds as chlorine or its derivatives. Confirmation of these theories must be sought in more extensive fundamental studies of the mechanism and rate of kill of organisms by various disinfectants, and of variations in these with changing and variable environmental factors.

It should be noted that just as there are differences in effectiveness of free chlorine forms, such as hypochlorous acid and hypochlorite ion, so also are there differences in disinfection efficiency among combined chlorine forms, such as the several inorganic or organic chloramines. Thus, regardless of what the mode of action of chlorine might be, it is abundantly evident that free and combined available chlorine forms are not alike with respect to disinfecting efficiency, for either the same or different microorganisms. Chlorination regimens in which hypochlorous acid is predominant are the most effective for disinfection.

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CHAPTER 4

# Chlorination Practices

Early water chlorination practices (variously termed "plain chlorination," "simple chlorination," and "marginal chlorination") were developed for the purpose of disinfection. As previously noted, chlorine-ammonia treatment (previously termed "chloramination") was soon introduced to *limit the development* of objectionable tastes and odors often associated with marginal chlorine disinfection. Subsequently, "superchlorination" (as then practiced) was developed for the additional purpose of *destroying* objectionable taste- and odor-producing substances often associated with chlorine-containing organic materials. The introduction of "breakpoint chlorination" and the recognition that chlorine residuals can exist in two distinct forms established contemporary water chlorination as one of two types: combined residual chlorination and free residual chlorination.

## Combined Residual Chlorination

Combined residual chlorination practice involves the application of chlorine to water to produce, with natural ammonia present or purposely added, a combined available chlorine residual and to maintain that residual through part or all of a water-treatment plant or distribution system. Combined available chlorine forms have lower oxidation potentials than free available chlorine forms, and are less effective as oxidants. They also are less effective disinfectants. In fact, about 25 times as much combined available residual chlorine as free available residual chlorine is necessary to obtain equivalent bacterial kills under the same conditions of pH, temperature, and contact time. And about 100 times longer contact is required for combined than for an equal amount of free available chlorine residuals to obtain equivalent bacterial kills under the same pH and temperature conditions.

When a combined available chlorine residual is desired, the characteristics of the water will determine how it can be accomplished: (1) If the water contains sufficient ammonia to produce, with added chlorine, a combined available



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chlorine residual of the desired magnitude, the application of chlorine alone suffices. (2) If the water contains too little or no ammonia, the addition of both chlorine and ammonia is required. (3) If the water has an existing free available chlorine residual, the addition of ammonia will convert the residual to combined available residual chlorine. A combined available chlorine residual should contain little or no free available chlorine.

The practice of combined residual chlorination is especially adaptable after filtration (posttreatment) for controlling certain algae and bacterial after-growths and reducing dead-end red-water troubles in potable water distribution systems, and for providing and maintaining a stable residual throughout the system to the point of consumer use. It frequently is preceded by free residual chlorination to ensure water potability, unless very long contact periods are available. (See Fig. 6.)

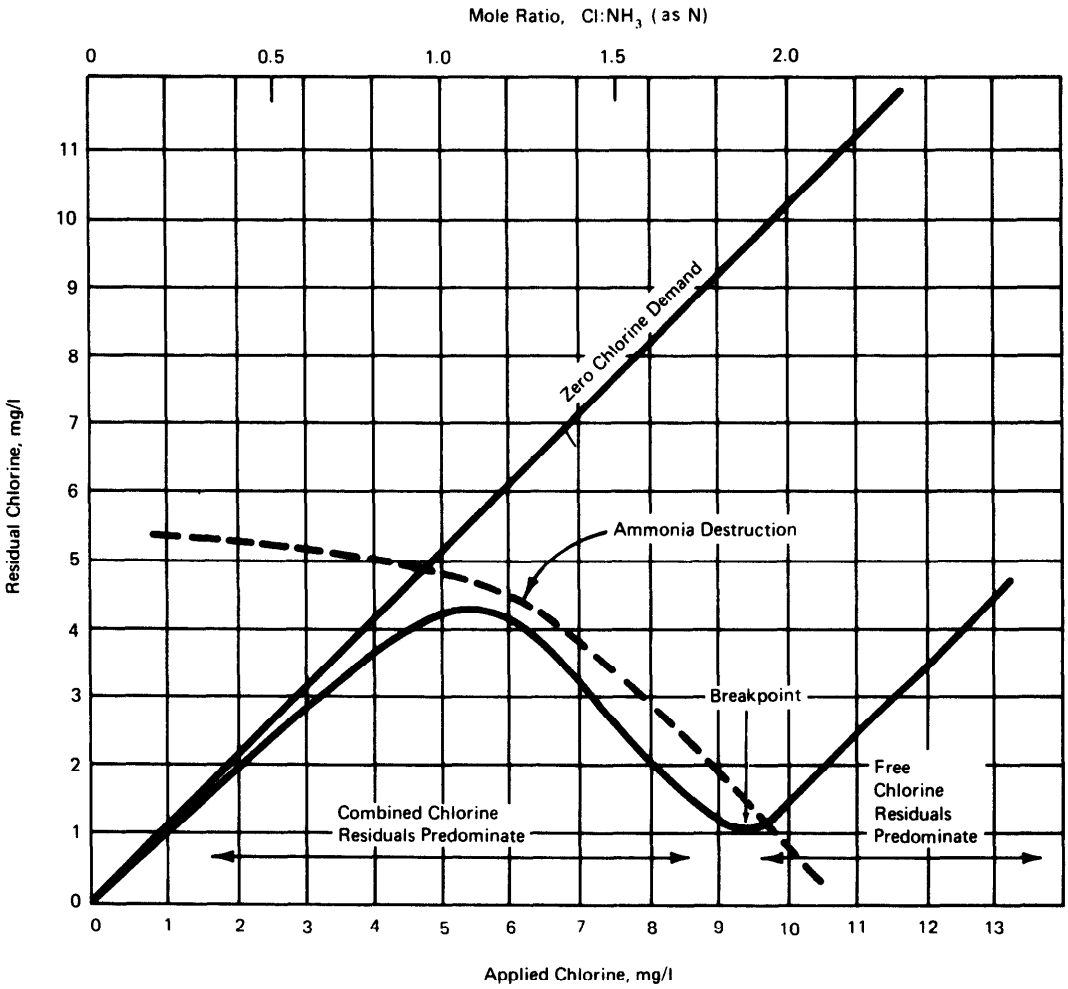


**Fig. 7.** Representative Breakpoint Chlorination Curve (N = 1 mg/1)

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## Free Residual Chlorination

Free residual chlorination practice involves the application of chlorine to water to produce, either directly or through the destruction of naturally present ammonia, a free available chlorine residual and to maintain that residual through part or all of a water-treatment plant or distribution system. Free available chlorine forms have higher oxidation potentials than combined available chlorine forms and are more effective as oxidants. Moreover, they are also more effective disinfectants, as already noted.

When free available chlorine residual is desired, the characteristics of the water will determine how it can be accomplished: (1) If the water contains no ammonia (or other nitrogenous materials), the application of chlorine will yield a free residual. (2) If the water does contain ammonia that results in the formation of a combined available chlorine residual, this must be destroyed by applying an excess of chlorine.

## Breakpoint Chlorination

With molar $C1:NH_3$ (as N) concentrations up to 1:1 (5:1 weight basis) monochloramine and dichloramine will be formed, the relative amounts of each depending on pH and other factors. As shown in Fig. 7, chloramine residuals generally reach a maximum at equimolar concentrations of chlorine and ammonia. Further increases in the $C1:NH_3$ ratio result in the oxidation of ammonia and reduction of chlorine. These oxidation-reduction reactions are essentially complete when two moles of chlorine have been added for each mole of ammonia present and if sufficient contact time is provided. Chloramine residuals then decline to a minimum value, the "breakpoint," when the molar $C1:NH_3$ ratio is about 2:1. At this point oxidation-reduction reactions are essentially complete. Further addition of chlorine produces free residual chlorine.

The mechanics and precise products of the breakpoint reactions have not been clearly established, and it is difficult to define precisely what occurs in different situations. When chlorine is added to water containing free ammonia, chloramines are formed, but as additional amounts of chlorine are applied the chloramines form intermediate compounds and eventually are destroyed as reactions go to completion. Thus, the chloramines first formed may exist while ammonia is in excess, but they are decomposed when chlorine is in excess. Beyond the breakpoint, unreacted hypochlorous acid remains in solution.

Equations Z-10 through Z-13 suggest the reactions that might occur in the destruction of chlorine (the true reactions probably encompass a combination of these and others).

$$2NH_3 + 3Cl_2 \longrightarrow N_2 + 6HCl \qquad \text{(Eq Z–10)}$$

$$4NH_2Cl + 3Cl_2 + H_2O \longrightarrow N_2 + N_2O + 10HCl \qquad \text{(Eq Z–11)}$$

$$2NHCl_2 + H_2O \longrightarrow N_2O + 4HCl \qquad \text{(Eq Z–12)}$$

$$HOCl + 2NHCl_2 + H_2O \longrightarrow 2NO_2 + 5HCl \qquad \text{(Eq Z–13)}$$

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In any event, it is likely that nitrogen, nitrous oxide, and perhaps some nitrogen trichloride are the principal gaseous products of the breakpoint reactions. In theory, the amount of chlorine required to arrive at the breakpoint (where all ammonia is oxidized) has been variously reported at between 7.6 to 15 times the ammonia nitrogen content of the water. In practice, however, due to the presence of organic and other chlorine reactive materials, up to 25 times as much chlorine as the ammonia nitrogen content may be required to reach this point. After free residual chlorination (beyond the breakpoint), the resulting residual should consist of at least 90 per cent free available residual chlorine and should contain little combined available residual chlorine.

Most of the present knowledge of the breakpoint process is attributable to Palin's work (c. 1950). His data, for a 10-min contact period and with C1:$NH_3$ (as N) ratios near the breakpoint of 8.6:1 (by weight), demonstrate that little free chlorine — and thus little HOC1 — is present at pH values less than about 7; also, that at pH values above about 7.3, almost half of the residual chlorine exists as free chlorine. A maximum of HOC1 relative to the dosage of chlorine occurs at pH near 7.5, rather than in the acid pH range expected (see Fig. 8) with aqueous chlorine solutions not containing ammonia.

Figure 8 illustrates this effect for a ratio of C1:$NH_3$ (as N), by weight, of 8.2 — just before the breakpoint. The fraction of remaining HOC1 to chlorine dose

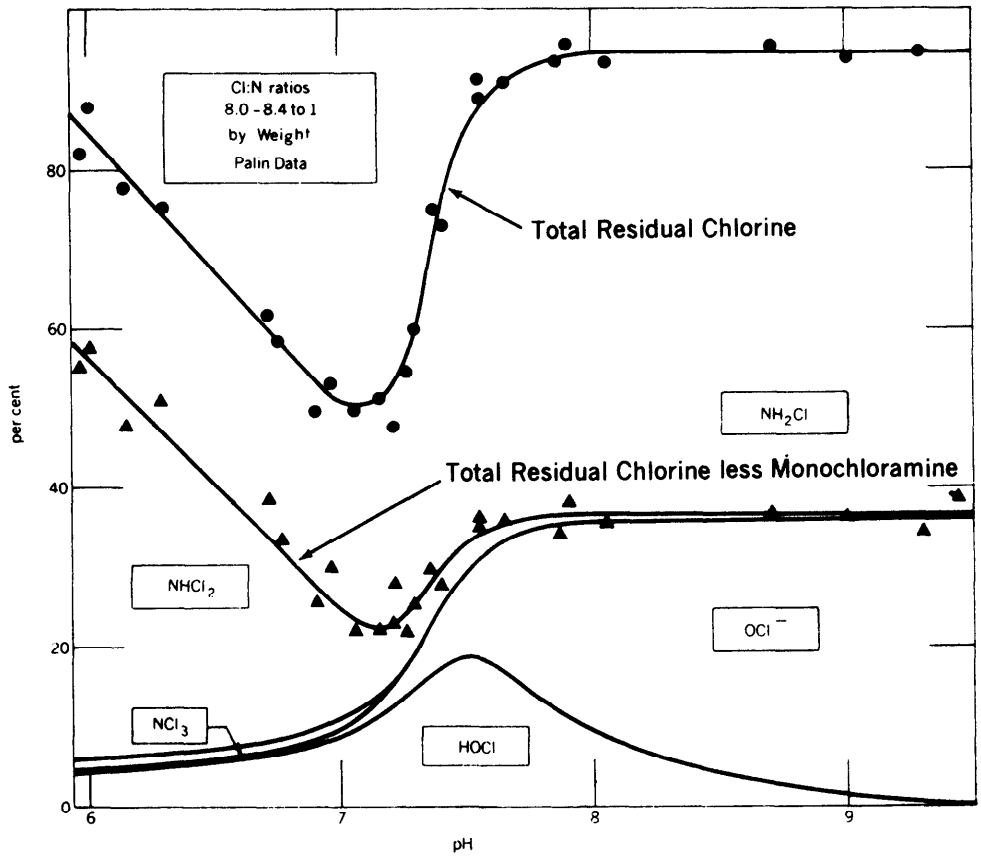


**Fig. 8.** Development of the Breakpoint with Chlorine-to-Nitrogen Ratio Equal to 8.2 ± 0.2

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at the maximum (pH near 7.5) is almost 20 per cent. Thus, if the initial $NH_3$ content were 0.5 mg/l (and thus the chlorine dosage were 4.1 mg/l; i.e., 8.2:1 = 4.1:0.5), at least 0.75 mg/l of HOC1 could be expected to be present for at least 10 min in the treated water. (About 18 per cent of 4.1 = 0.75 mg/l.)

Figure 9 illustrates this effect for a ratio of C1:$NH_3$ (as N), again by weight, of 12, which is well beyond the breakpoint. (Note that there is an excess of HOC1 present at all times, even in acid solution.) The fraction of remaining HOC1 to chlorine dose at the maximum still exists near pH of 7.5 and, in this case, is about 25 per cent of the original chlorine dosage. Thus, again if the initial $NH_3$ content were 0.5 mg/l (and then the chlorine dosage were 6 mg/l; i.e., 12:1 = 6:0.5), at least 1.5 mg/l of HOC1 could be expected to be present for at least 10 min in the treated water (about 25 per cent of 6 = 1.5 mg/l). After the breakpoint about 1.5 mg/l of free chlorine, or about 0.8 mg/l of HOC1, would remain. (Good virucidal effects should result in both cases.)

Free residual chlorination or breakpoint chlorination is especially suitable in the following cases:

1. Where raw water quality is poor and inactivation of enteric viruses is indicated
2. For oxidation of iron and manganese
3. When contact time is insufficient to accomplish disinfection by combined available chlorine forms

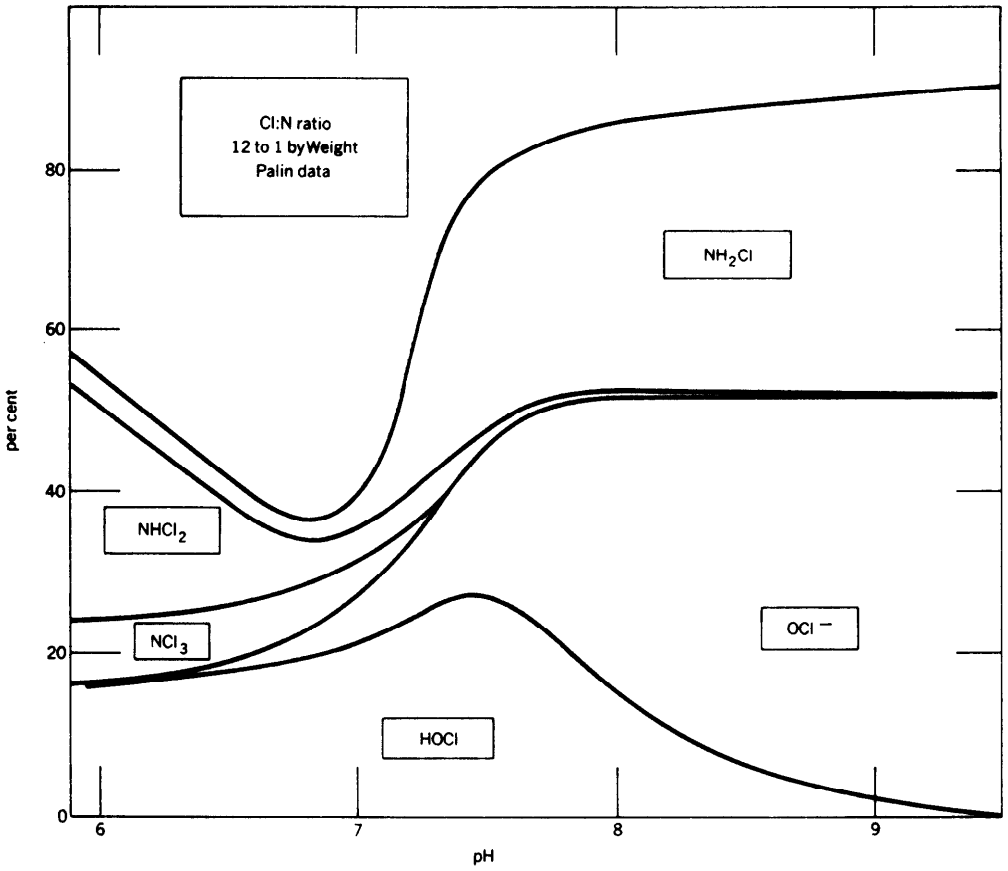


**Fig. 9.** Development of the Breakpoint with Chlorine-to-Nitrogen Ratio Equal to 12

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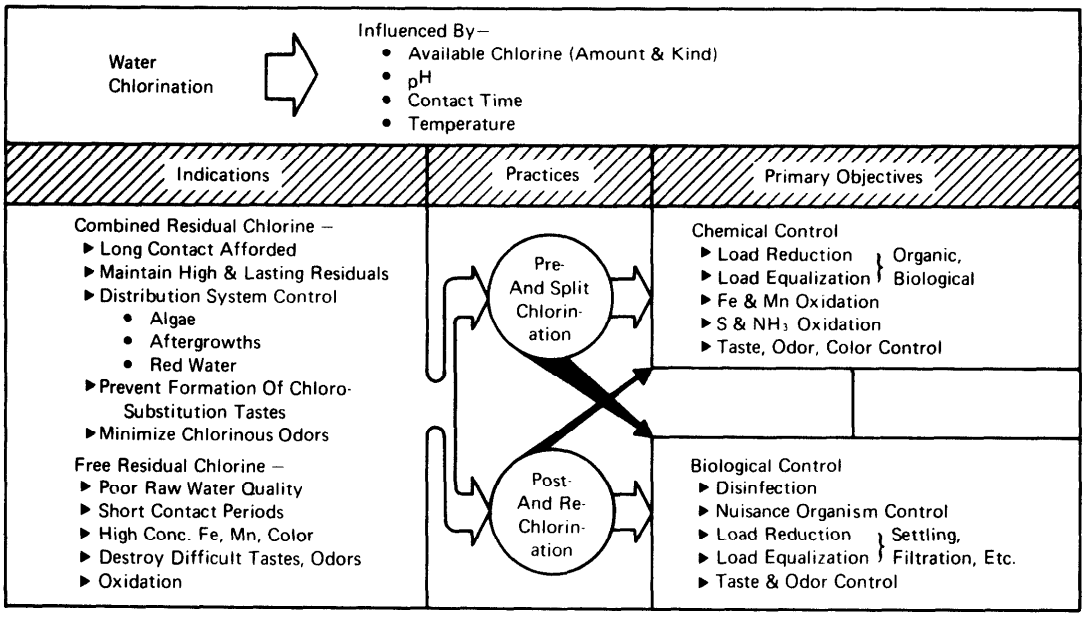


**Fig. 10.** Summary of Principles of Conventional Water Chlorination

4. When tastes exist before treatment (also to destroy objectionable tastes and odors produced by some substances that react with chlorine and by certain algae growths)

5. To minimize biological growths on filters and lengthen filter runs

6. To eliminate chlorine-resistant bacteria and aftergrowths in distribution systems

The practices are adaptable to pre-, post-, and rechlorination schemes, and when chlorination is the only treatment process employed; see Fig. 10.

## Biological Control

Data are unavailable to provide a specific regimen for achieving 100 per cent kill of all microorganisms of sanitary significance under all conceivable combinations of each of the influencing variables already discussed. Contemporary chlorine disinfection is based on the attainment of a distinct *kind* and *amount* of residual chlorine remaining after the treatment that has been established to be adequate to ensure water potability.

Water-borne organisms of particular concern are those etiologically associated with typhoid fever, dysentery, and various gastrointestinal disorders. The causative organisms associated with most of these are of the genus *Salmonella* or *Shigella*. Tuberculosis, anthrax, tularemia, and streptococcal or other bacterial infections in man are claimed by some to be transmitted by polluted water, but firm, supporting epidemiological data are lacking; moreover, the causative organisms do not often exist nor do they normally survive long in water.

Effectiveness of disinfection is determined by testing for the presence of coliform organisms. This conclusion is based, in part, on the fact that the causative bacterium of typhoid fever, *S. typhosa*, is at least as vulnerable as

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most coliforms and, further, that its vulnerability is fairly representative of *Salmonella* and *Shigella* water-borne pathogens.

Of protozoan diseases, involving parasitic organisms that invade the body via the digestive tract, amebic dysentery is by far the most important. Although the low incidence of this disease provides some degree of public health assurance, it is important to recognize that at concentrations normally employed by water utilities, chlorine is not effective in the destruction of the causative *E. histolytica* cyst. Accordingly, the coliform standard of bacteriological safety is of no value in appraising water potability from this standpoint where chlorine disinfection is the only treatment provided.

Enteroviruses comprise a group of organisms that multiply in the human intestine and are discharged in feces; they include several types of polio virus, Coxsackie virus, and ECHO (enteric cytopathogenic human orphan) viruses. The etiological significance of such viruses in water, resulting from pollutional discharges, has not been fully established; however, it is important to note that water-borne outbreaks of some virus diseases might occur if the raw water is grossly polluted.

Of all known viruses, that associated with infectious hepatitis is of greatest present concern. Reports of epidemics have been recorded that indict drinking water as the vehicle of disease transmission. With a few exceptions, however, these epidemics have involved untreated supplies or treated supplies subsequently contaminated in the water-distribution system. Polio virus has frequently been mentioned as being possibly water-borne. Several epidemics of poliomyelitis attributable to water have been cited, but evidence has not been sufficiently complete to carry conviction. The possibility that polio is occasionally water-borne is not excluded, but proof is lacking. Additionally, there are the adenoviruses, which comprise a group of respiratory viruses that multiply in human beings and are discharged in feces. No outbreaks of water-borne adenovirus infection have been documented.

Although there is cause for concern over the role of water-borne viruses and chlorine-resistant organisms in disease transmission, sufficient evidence is not yet available to negate existing control practices totally. It is clear that enteric viruses differ in sensitivity and resistivity to chlorine. It also is clear that viruses are more resistant to chlorine than are coliforms and other enteric bacteria.

Recently there also has been concern about nematodes in water. Free-living, nonpathogenic varieties are sometimes present in surface water supplies, but, although these organisms are capable of ingesting and harboring other potentially dangerous organisms, they are not known to have been involved in disease transmission. From an esthetic viewpoint, however, their presence in water is undesirable. Nematodes exhibit an even higher resistance to chlorine than do enterobacteria and enteroviruses.

**Minimum bactericidal residuals.** Minimum bactericidal chlorine residuals were investigated in the 1940s by the Public Health Service. Data secured in experiments at temperatures from 20C to 25C, using 10-min exposures to free available chlorine and 60-min exposures to combined available chlorine, are the basis for recommended minimum safe residuals illustrated in Fig. 11. Thus, with free available residual chlorine at pH 6.0 to 8.0, a safe residual for complete

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destruction of bacteria after l0-min contact would be not less than 0.2 mg/l; at pH 8.0-9.0, at least 0.4 mg/l; at pH 9.0-10.0, at least 0.8 mg/l; and, above pH 10.0, more than 1.0 mg/l would be required. With combined available residual chlorine at pH 6.0-7.0, a safe residual after 60.min contact would be not less than 1.0 mg/l; at pH 7.0-8.0, at least 1.5 mg/l; and at pH 8.0-9.0, at least 1.8 mg/l. These values are not applicable to all waters, particularly those that are turbid, nor do they apply at temperatures below 20C.

In 1956, the National Research Council, in investigating minimum chlorine

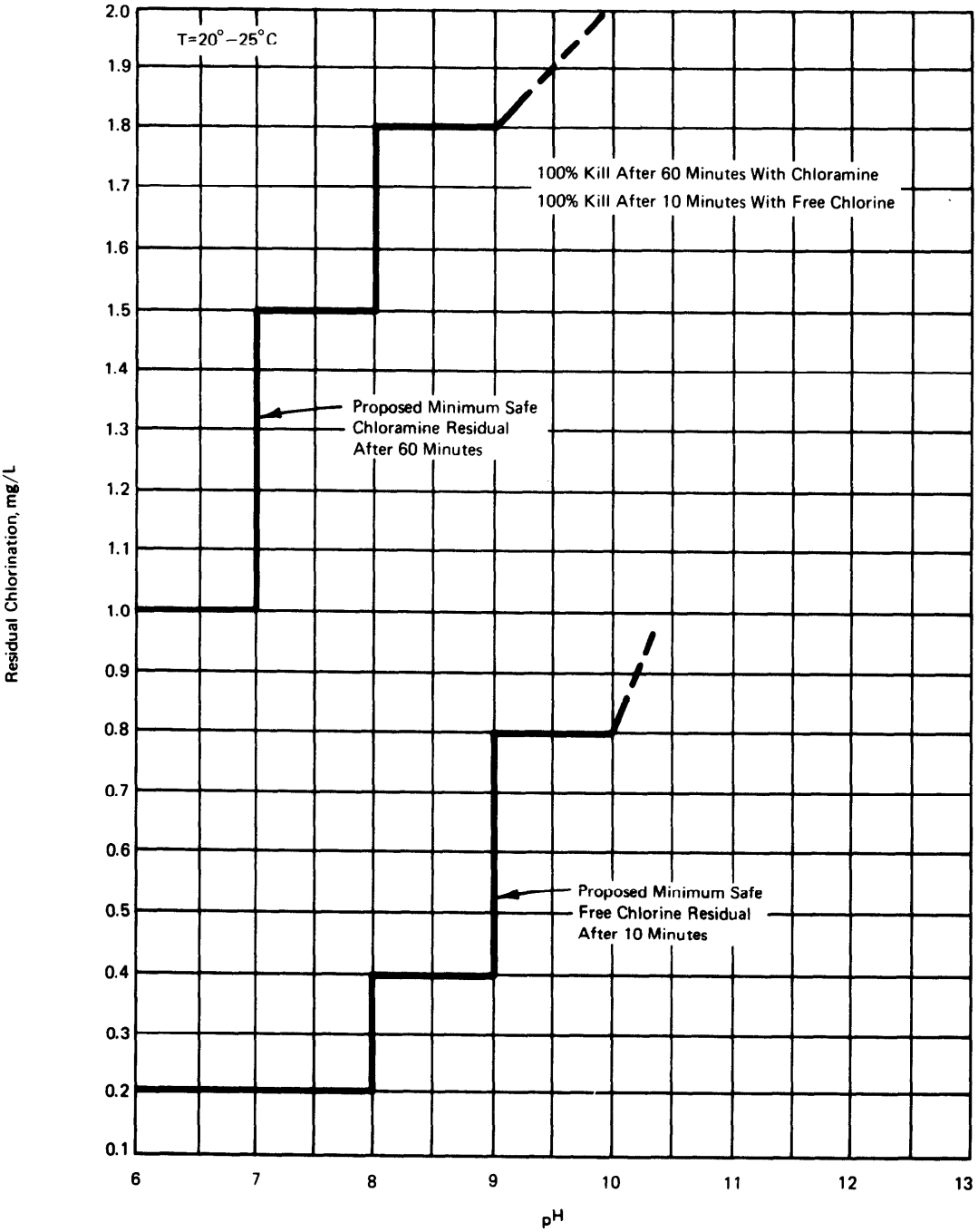


**Fig. 11.** Minimum Free and Combined Bactericidal Chlorine Residuals

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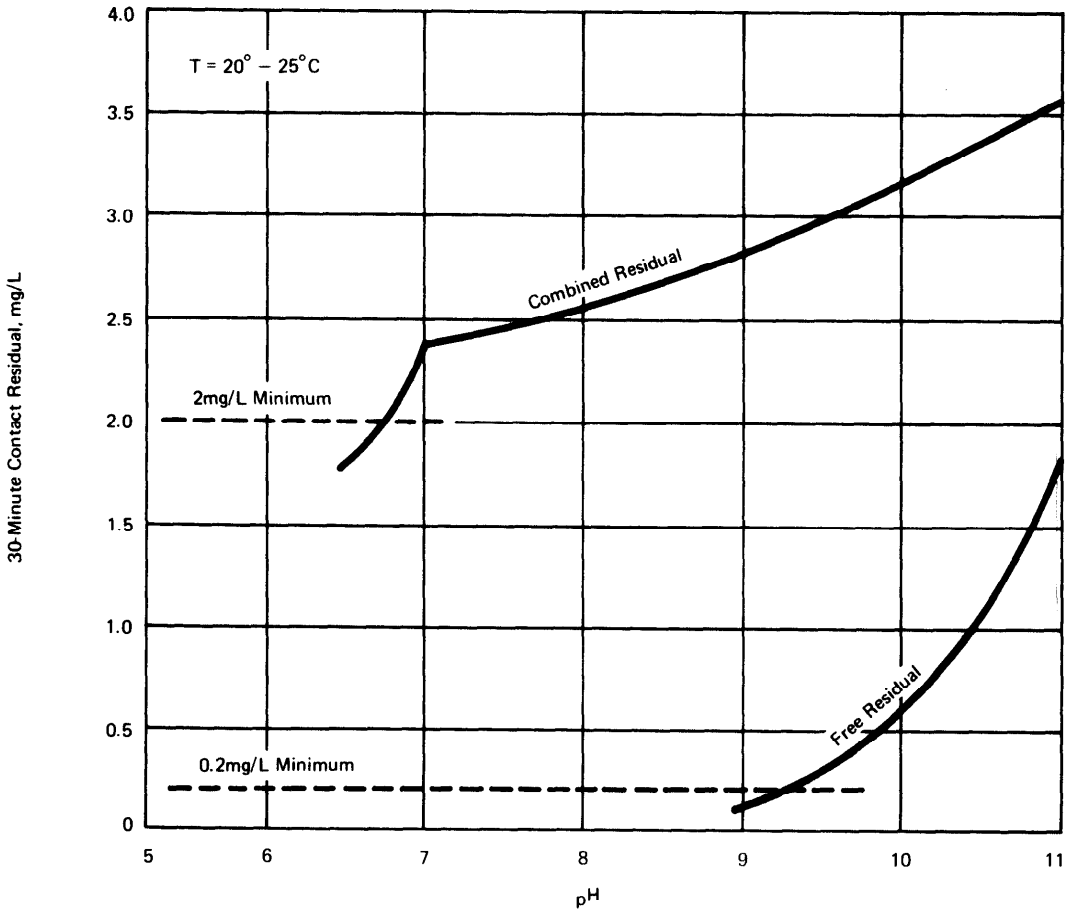


**Fig. 12.** Minimum Free and Combined Bactericidal Chlorine Residuals (Based on 30-Min Contact Periods)

residuals applicable for military water supplies, reanalyzed the Public Health Service data and submitted recommendations based on 30-min contact periods. As illustrated in Fig. 12, with free available residual chlorine up to pH about 9.2, a safe residual for complete destruction of bacteria would be not less than 0.2 mg/l; at pH 10.0 it would be at least 0.6 mg/l. A combined available residual of 2 mg/l is indicated where pH is 6.8 or below, and at pH 10.0 more than 3 mg/l might be necessary.

**Minimum cysticidal residuals.** Minimum recommended cysticidal chlorine residuals are illustrated in Figs. 13 and 14. It should be noted that minimum chlorine requirements for cyst destruction are greater than for bacteria destruction and that diminished cystical effect at reduced temperatures is more pronounced even under the best conditions of operation. Thus, more than 50 mg/l free available residual after 30-min contact would be necessary when the pH is 8.3 and 9.5, respectively, at 2-5C and 22-25C. Waters containing significant amounts of organic matter, or high bacterial or cyst densities, would require either higher 30.min residuals or longer contact periods than those illustrated.

**Virucidal residuals.** It is becoming increasingly evident that the probability of transmission of viral disease by water is great enough to warrant special

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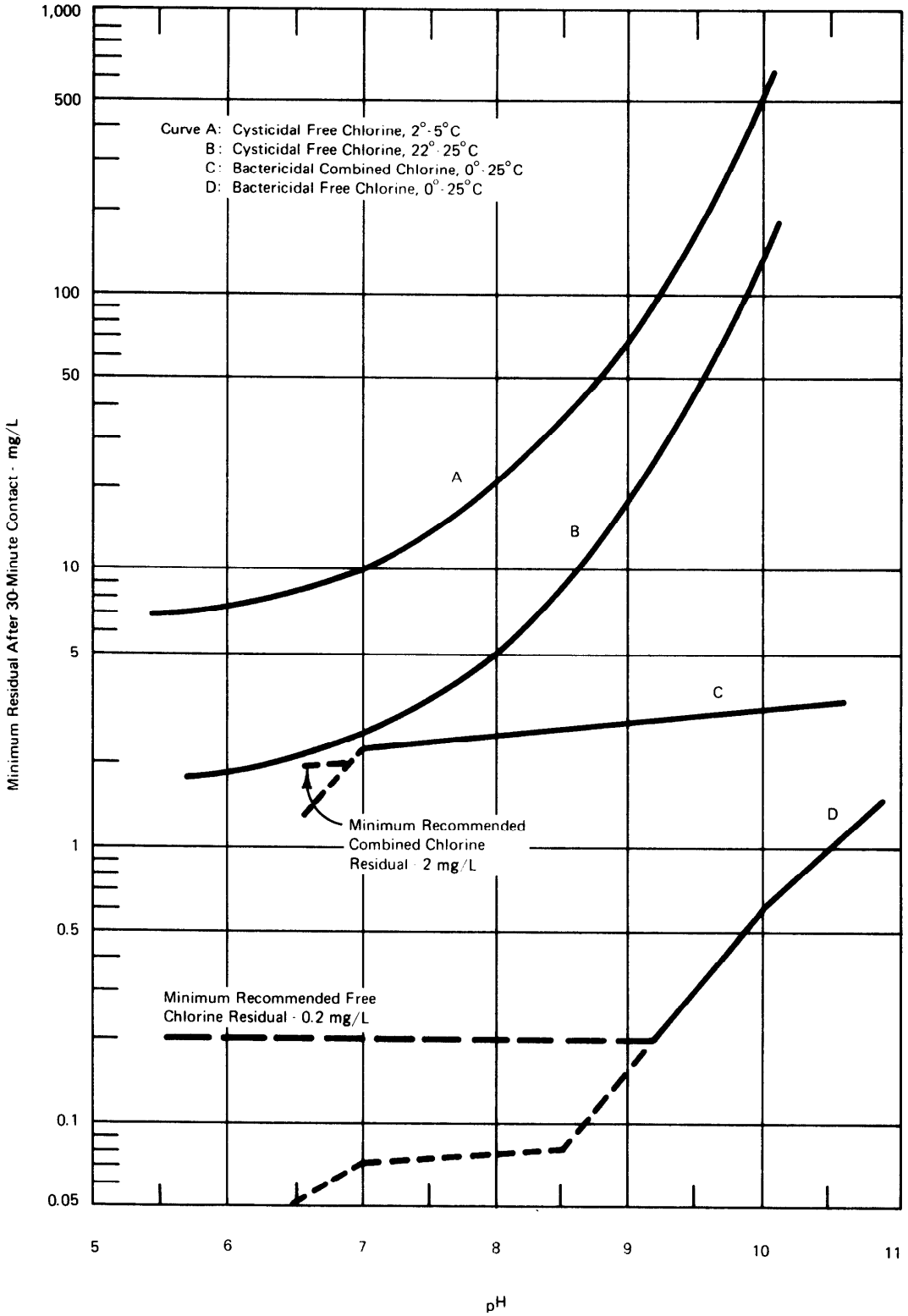


**Fig. 13.** Minimum Free and Combined Bactericidal and Cysticidal Chlorine Residuals

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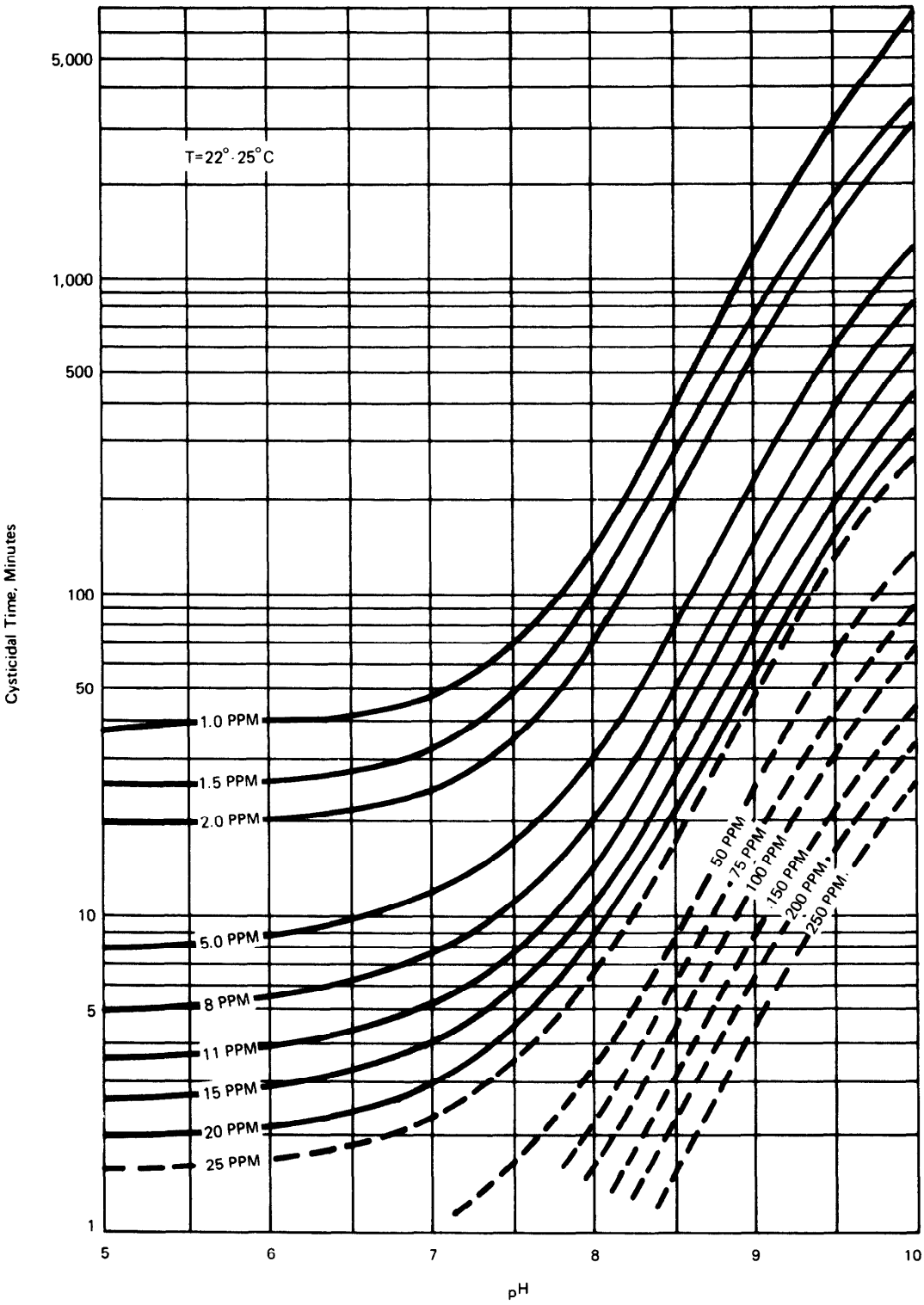


**Fig. 14.** Minimum Contact Time for Free Residual Chlorine Destruction of *E. Histolytica* (ppm = mg/l)

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attention whenever the raw water is subject to more than occasional opportunity for contamination. Thus, any water source that regularly receives municipal or other human wastes should be regarded as contaminated with viruses until it has had a minimum of 30 days of protected storage. Disinfection of such waters should be such as to provide inactivation of any viruses present. Suitable antiviral disinfection should nearly sterilize the water; the total bacteria count should not exceed one per milliliter and possibly not more than one per 10 ml.

Of all common forms of aqueous chlorine, only undissociated hypochlorous acid, HOC1, can be considered an effective antiviral agent. Numerous studies have demonstrated that about 25 mg/l of chloramine is required for adequate viral inactivation of the most resistant enteric viruses studied; whereas more than 100 mg/l of hypochlorite, $OC1^{-}$, is required for equivalent effects. In contrast, only 0.5-l.0 mg/l of hypochlorous acid will provide effective viral inactivation within 30 min. Thus, free residual chlorination in a pH range yielding substantial HOC1 is called for when potential contamination by viruses exist. Moreover, because such waters are likely to contain substantial amounts of ammonia-nitrogen, breakpoint chlorination is indicated. (HOC1 formed during the development of breakpoint as well as that formed upon completion of the reactions is available for virus inactivation.)

For waters containing from a few tenths to 1 mg/l of ammonia-nitrogen, the best pH for free residual chlorination designed to yield water free of viruses is near pH 7.5. With such a regimen, a high fraction of HOC1 will exist during the critical disinfection period, and there will also be much less tendency for the formation of nitrogen trichloride than in more acid waters.

It should be noted that when such chlorination regimens are used, the standard coliform test loses much of its indicator value for operation of the disinfection process. Negative coliform tests will be obtained even when viral destruction is not adequate. Research is needed to develop a more rugged indicator organism.

The adoption and maintenance of fixed, minimum free or combined available chlorine residuals serve principally as *operating guides* and cannot completely replace biological or microscopic analyses. Demonstration of the absence of disease-producing organisms still is the only positive means of assuring water potability.

## Points of Application

When chlorination of water was first employed for disinfection, terminal treatment of the plant effluent was almost invariably practiced. Now the use of chlorine in various stages of water treatment, and even in the distribution system, is common practice. Multiple or split chlorination schemes are increasingly employed for disinfection and other purposes and frequently enhance the efficiency of many-unit water-treatment processes. Fundamentally, the points at which chlorine is applied depend on the specific objectives of chlorination — technical, practical, safety, economic, and other considerations. In practice, various terms have come into common usage to specify the point and location at which chlorine is applied.

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**Plain or simple chlorination** involves the application of chlorine to water that receives no other treatment. More than half of all existing water-treatment facilities in the US fall in this category; thus, this treatment represents their sole public health safeguard. When applied to impounded or naturally elevated surface systems, chlorine usually is injected to the pipeline leading from such a gravity supply. When applied to water pumped into a system, chlorine usually is added at the pump suction, using pressure beyond the pump to operate the chlorine feeders.

**Prechlorination** involves the application of chlorine to water prior to any other unit treatment process. Among benefits that can be achieved are improved filter operation by reduction and equalization of the bacterial and algal load and by control of slime and mud ball formation; improved coagulation; reduction of taste-, odor-, and color-producing materials by oxidation and retardation of decomposition (in settling units); and the provision of a safety factor in disinfecting heavily contaminated waters while keeping the chlorine residual in the distribution system at a minimum. When prechlorination is employed it usually is desirable to obtain a contact period that is as long as possible, preferably by applying chlorine to the raw water suction intake to provide contact during the entire purification process. The dosage, of course, depends on the objective. In some cases free available chlorine residuals are indicated; in others, combined available chlorine residuals may suffice. Care must be exercised to maintain the proper residual necessary to accomplish the desired objective.

**Postchlorination** involves the application of chlorine to water following any other unit treatment process. The most important form of postchlorination is that following filtration for disinfection, to provide either a free or a combined residual chlorine in a part of or the entire distribution system. The contact period provided to effect disinfection, as discussed elsewhere, is an important consideration; chlorine is usually added to the filter effluent. When post-chlorination follows prechlorination and precedes filtration, greater filter efficiency usually is obtained. The second addition of chlorine raises the chlorine residual in the delivered water to a predetermined amount and thus reestablishes the portion of chlorine that had been consumed in the purification process.

**Rechlorination** involves the application of chlorine to water, following previous chlorination treatment, at one or more points in the distribution system. The practice, which may involve free or combined residual chlorination, is especially common where the distribution system is long and complex and where the plant effluent residual is insufficient to control bacterial and algal regrowths, red-water troubles, etc. The chlorine may be applied at the end of a long main in the distribution system, at a point where a main supplies water to an outlying community, or at such places as a reservoir, standpipe, or booster pumping station.

If a long contact period is available, combined available residual chlorine treatment may be employed as plain chlorination, prechlorination, post-chlorination, or rechlorination. The points of application of ammonia and chlorine should be sufficiently far apart to permit thorough mixing of the ammonia with the water before chlorine is applied, but not so distant as to permit dissipation of ammonia by organic matter before chlorine is added. The

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earlier addition of chlorine may result in the formation of chlorophenols and other objectionable products. If a long contact period is not available, the general trend is to obtain free residuals through prechlorination and, later in the course of treatment, to convert the residual to a combined form that persists throughout the distribution system.

## Dechlorination

When combined residual chlorination or free residual chlorination is, employed, chlorine residuals of a certain type and intensity must be attained to accomplish a particular objective. In some instances the residual might be of such magnitude that the water will be esthetically or otherwise objectionable, and a portion of the residual will have to be removed. *Dechlorination is the partial or complete reduction of residual chlorine in water by any chemical or physical treatment.* (Lately there has been considerable concern about the toxic effects of chlorine, especially as chloramine, on fish that come in contact with chlorine-treated wastewater discharges.)

The origin of superchlorination-dechlorination applications already has been discussed. In the original applications, superchlorination was loosely understood to involve concentrations of chlorine greater than that required to provide the chlorine residual desired in the type of treatment now termed combined residual chlorination. No recognition was given to the *type* of residual obtained, for differential residual tests had not yet been developed. In some instances and under some conditions, combined available chlorine residuals resulted from such treatment; in other instances free available residuals resulted. The residuals were, however, sufficiently high to warrant and require dechlorination. Contemporary applications of the superchlorination-dechlorination concept are more suitably described as high free residual chlorination followed by dechlorination. Such applications are especially suitable where available contact time is very limited; they usually involve prechlorination in a water-treatment plant or plain chlorination in a pipeline, followed by dechlorination before the water reaches the distribution system.

Dechlorination by sulfur dioxide and its derivatives is feasible, rapid, and precise. These materials, often called dechlors, function as reducing agents and may be applied ahead of filtration in the clear well or following plain chlorination. Sulfur dioxide is the most widely used dechlor. Sodium bisulfite and sodium sulfite also are effective, but the former is ordinarily used in practice because it is cheaper and more stable. Sodium thiosulfate, in either crystalline or solution form, is commonly used to dechlorinate water samples intended for bacteriological analysis; its application on a plant basis appears to be limited to certain industrial water treatment processes. [It is also the standard titrating agent in the iodometric technic for determining residual chlorine (see Eq Z-20).] Table 2 illustrates the chemistry involved for the various dechlors.

Aeration of chlorinated water by submerged or spray aerators will lead to release of some residual chlorine. In the case of free available residuals, reduction is limited because calcium or sodium hypochlorite formed by the reaction of HOC1 with natural alkalinity of the water is nonvolatile. Nitrogen trichloride, if formed, and free chlorine ($C1_2$) can be readily volatilized by any

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type of aeration. At low pH levels (about 5) combined available residuals exist mostly as dichloramine, which is substantially reduced by aeration. At high pH levels (about 9), where monochloramine is predominant, very little loss of residual can be effected by aeration. Prolonged storage, especially in the presence of sunlight, also will result in the gradual disappearance of residual chlorine.

Controlled ammoniation has been used to some extent for dechlorinating HOC1 residuals. It has been observed that 1.0 mg/l HOC1 can, in accord with theory, be effectively dechlorinated by 0.15 mg/l $NH_3$. The method appears to be ineffectual for monochloramine particularly and, to a lesser extent, for dichloramine reduction in the presence of HOC1, whereas other materials such as sulfur dioxide are effective.

### Engineering Considerations

In addition to chemical considerations pertinent to efficient disinfection, various engineering considerations are important. First, a complete, uniform mix of chlorine with water to be disinfected must be achieved as rapidly as possible. Substantial kills (perhaps 90 per cent or better) of bacteria and viruses may be obtained within very short times before added chlorine has had a chance to react with nitrogenous materials present. Rapid, uniform mixing promotes rapid kills, whereas imperfect mixing tends to facilitate full formation of chloramines in some parts of the water before there is contact with the organisms in other parts. Considerable improvement in disinfection efficiency can be anticipated with more rapid mixing units.

The design of holding or chlorine contact tanks also is important. Disinfection is one process for which any measurable short-circuiting is completely ruinous to process efficiency. (A direct leakage of as little as 0.01 per cent of raw water may cause the water to fail to meet minimum hygienic standards.) Attention to tank shape and to proper baffling is critical.

**TABLE 2**
**Sulfur Dioxide and Other Materials for Dechlorination**

| Dechlorinating Agent | Parts Required per Part Chlorine Removed (Theoretical) | Alkalinity Consumed (as $CaCO_3$) | Equation* |
|---|---|---|---|
| Sulfur dioxide, $SO_2$ | 0.9 | 2.8 | Z-14 |
| Sodium bisulfite, $NaHSO_3$ | 1.46 | 1.38 | Z-15 |
| Sodium sulfite, $Na_2SO_3$ | 1.77 | 1.38 | Z-16 |
| Sodium thiosulfate, $Na_2S_2O_3$ | 0.7 | – | Z-17 |
| Activated carbon, C | 0.085 | 2.1 | Z-18 |

*$SO_2 + Cl_2 + 2H_2O = 2HCl + H_2SO_4$ (Eq Z-14)
$NaHSO_3 + Cl_2 + H_2O = NaHSO_4 + 2HCl$ (Eq Z-15)
$Na_2SO_3 + Cl_2 + H_2O = Na_2SO_4 + 2HCl$ (Eq Z-16)
$2Na_2S_2O_3 + Cl_2 = Na_2S_4O_6 + 2NaCl$ (Eq Z-17)
$C + 2Cl_2 + 2H_2O = 4HCl + CO_2$ (Eq Z-18)

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CHAPTER 5

# Distribution System Chlorination

## New Mains

Special care should be taken to insure disinfection of new mains. Among possible sources of contamination are sewer drainage, contaminated soil in the trench, contamination from workmen and/or their equipment, and, of course, unavoidable foreign material present in the trench during construction.

Education of crew members as to the need for *avoiding* contamination of the main during construction is fundamental. Contractors and workmen should be thoroughly familiar with all pertinent state and local requirements governing installations of mains. All sewers, water mains, and other underground conduits should be located prior to construction and relocated, if necessary, to prevent contamination during construction. Pipe should be strung on high ground. At all times when construction is not actually in progress, watertight plugs should be installed in all pipe openings. (Gunny sacks and rags are not adequate.) Provision should be made to pump any water that might collect in the trench. Special care should be taken to avoid contamination of valves, fittings, and pipe interiors, both before and during construction; each should be inspected and, if necessary, cleaned before installation.

After pressure-testing the main, it should be flushed with water of sufficient velocity to remove all dirt and other foreign material. When this process has been completed, disinfection (using liquid chlorine, sodium, or calcium hypochlorite) can proceed by one of AWWA's recommended methods next described.

**Continuous feed.** In this method, water from the distribution system or other approved source and the chlorine form selected are fed at constant rate into the new main at a concentration of at least 50 mg/l. A properly adjusted hypochlorite solution injected into the main with a hypochlorinator, or liquid chlorine injected into the main through a solution-feed chlorinator and booster pump, may be used. The chlorine residual should be checked at intervals to



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insure that the proper level is maintained. Chlorine application should continue until the entire main is filled. The water should remain in the main for a minimum of 24 hr, during which time all valves, hydrants, etc., along the main must be operated to insure their proper disinfection. Following the 24-hr period, no less than 25 mg/l chlorine residual should remain in the main.

**Slug method.** In this method a continuous flow of water is fed with a constant dose of chlorine (as in the previous method) but with rates proportioned to give a chlorine concentration of at least 300 mg/l. The chlorine is applied continuously for a period of time to provide a column of chlorinated water that will contact all interior surfaces of the main for a period of at least 3 hr. As the slug passes tees, crosses, etc., proper valves must be operated to insure their disinfection. This method is used principally for large-diameter mains where continuous feed is impractical.

**Tablet method.** Because this method precludes preliminary flushing, it is considered the least satisfactory, although it is commonly used for small-diameter mains. Calcium hypochlorite tablets are placed in each section of pipe, hydrants, and other appurtenances. The main then is slowly filled with water at a velocity of less than 1 ft/s to prevent washing of the hypochlorite to the end of the main. The final solution should have a residual of at least 50 mg/l and should remain in contact for a minimum of 24 hr.

Regardless of the method used, it is necessary to make certain that backflow of the strong chlorine solution into the supplying line does not occur. Following the prescribed contact period, the chlorinated water should be flushed to waste until the remaining water has a chlorine residual approximating that throughout the rest of the system. Bacteriological tests as prescribed by the authorities should be taken, and if the results fail to meet minimum standards, the disinfecting procedure must be repeated and the results again tested before placing the main in service.

## Repaired Mains

Broken mains represent one of the greatest problems in water utility disinfection practice. The very nature of a main break results in insanitary conditions in the area of the break. Moreover, the necessary haste in repairing the break increases the problems.

Following location and exposure of the break, the area should be carefully inspected to locate and isolate sewers and other contaminant sources. If time is available, it is advisable to flush the main while standing water in the trench is being pumped away. This procedure will remove much of the contamination that has flowed back into the main.

Minimum disinfection involves swabbing the new pipe sections and fittings with a 5 per cent hypochlorite solution before installation, and flushing the main — from both directions, if possible — before returning the system to service. Where conditions permit, the affected section should be isolated, service connections should be shut off, and the section should be subjected to disinfection procedures described for new mains. The slug may be increased to 500 mg/l chlorine and the contact time reduced to 30 min if essential.

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Bacteriological samples should be taken following disinfection to test the effectiveness of the process.

## Tanks and Reservoirs

Policies vary widely, but a schedule of tank and reservoir inspection, cleaning, and disinfection should be established and followed. Several disinfection procedures can be used before restoring the system to service. One method involves the addition of chlorine directly to the filling water. Sodium hypochlorite, for example, can be added via a portable hypochlorinator, or calcium hypochlorite powder can be added to the fill water by simply distributing it evenly on the surface (or by predissolving it in water and by spraying the water surface). There is no general agreement on the disinfection regimen, but 50 mg/l residual chlorine maintained for a minimum of 6 hr is widely employed.

Alternatively, liquid chlorine, applied as a water solution through a solution-feed chlorinator, may be added to the filling water. The chlorine may be added continuously during the filling process to impart a 50 mg/l residual, or a heavy dose may be added initially, with the tank then being filled to provide the desired residual. This approach obviously requires the use of portable chlorination equipment as well as chlorine cylinders and the necessary fittings and appurtenances.

Other methods may be used. For instance, the walls and other surfaces of the tank or reservoir may be sprayed with a solution containing a chlorine concentration of about 500 mg/l to provide almost immediate disinfection without harm to the tank or reservoir coating.

## Well Disinfection

As with water mains and reservoirs there is danger of contamination from equipment, materials, and surface water during the construction or repair of wells. Disinfection is necessary. Regardless of the form of chlorine employed, it is usual to provide a 50-100-mg/l free available chlorine residual for 12-24 hr to insure adequate disinfection.

To disinfect a dug well, the casing or lining should be washed thoroughly with a strong chlorine solution (about 50 mg/l). Water should then be pumped from the well until clear; then the temporary pumping equipment should be removed and the well cover installed. Prior to inserting the pump cylinder and drop-pipe assembly, the disinfectant should be added through a pipe or hose to a minimum of 50 mg/l. The pump cylinder and drop pipe then should be washed with a strong chlorine solution before installation. The chlorine solution should then be allowed to remain for 12-24 hr before it is pumped to waste in order to remove excess chlorine.

Disinfection of drilled, driven, or bored wells may be accomplished in a similar fashion. The test pump should be operated until the water is practically free of turbidity before test equipment is removed. Then a chlorine solution containing a minimum of 50 mg/l of a volume equal to at least twice the volume of water in the well is added to carry the solution into the voids of the water-bearing formation. Pump cylinders and drop pipe are washed with a strong

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chlorine solution as the assembly is lowered into the well. After 12-24 hr of contact, the chlorinated water should be pumped to waste.

In the case of a flowing or artesian well, it is necessary to restrict the continuous flow of water in order to maintain a sufficient chlorine residual. The chlorine solution should be added to the bottom of the well by using the same general procedures described for drilled wells to provide as much diffusion as possible. The flow from the well should then be sufficiently throttled to produce a residual of about 50 mg/l at the well surface. The valve should be closed completely for 12 hr or more before it is opened and the water discharged to waste. Refer to the latest revision of AWWA Standard A100 for additional details.

## Precautions

Field use of chlorination chemicals and equipment presents particular potential personnel hazards, and all pertinent safe handling practices must be adhered to. In addition, special precautions should be taken to prevent damage to aquatic life in receiving waters from heavily chlorinated waters. Federal, state, and local regulations regarding toxic wastes must be followed. If necessary, dechlorination of the wastewater should be provided prior to discharge.

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CHAPTER 6

# Chlorination Control

## Chlorine Demand

The chemical characteristics of water sources differ tremendously, as do, perhaps to a lesser degree, those of a single source from time to time. The amounts of chlorine necessary to accomplish a desired objective clearly reflect these differences. Thus, effective chlorination control necessitates adjustments in chlorine feed not only to allow for the variation in water flow, but also to compensate for variations in water quality. Principal among chlorine-reactive materials of concern in water utility practice are inorganic reducing substances, ammonia, amino acids, proteins, and carbonaceous material. These materials consume the chlorine supplied, and some of the reactions may occur before disinfection is accomplished.

Materials in water that influence the chlorine demand complicate the use of chlorine for disinfection because sufficient chlorine must be applied not only to destroy organisms, but to compensate for the chlorine consumed by these reactions. Reactions within organic materials generally are very rapid and stoichiometric; reactions with organic materials generally are slow, and their extent depends upon the concentration and kind of chlorine present.

Where modification of the chemical characteristics of water is a desirable adjunct of chlorine disinfection, such as to control tastes and odors of organic origin, the chlorination regimen is especially critical. For example, when a sufficient dose of chlorine is applied to yield a detectable residual, it will combine with organic materials and the residual will be predominantly combined available chlorine. As more chlorine is applied to obtain a higher residual, the chlorine demand will be increased and the remaining residual may include some free available chlorine. If sufficient chlorine is applied to yield a residual that is predominantly free available chlorine, oxidation reactions will prevail. Thus, instead of chlorine combining with organic materials that might



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cause undesirable tastes and odors, it may completely oxidize them, destroying or changing them into less complex substances that do not cause disagreeable tastes and odors.

*Chlorine demand is defined as the difference between the amount of chlorine applied to water and the amount of free, combined, or total available chlorine remaining at the end of a specified contact period.* It expresses a definite equilibrium of chemical reactions of chlorine in water under known conditions. For achieving the desired equilibrium of chemical reactions, it is necessary to allow sufficient time and to provide an unreacted residual, which may consist of free or combined available chlorine, or both. For a particular water, the chlorine demand will vary with the amount of chlorine applied, the time of contact, and the temperature. Determination of its magnitude must take all these factors into consideration. *Standard Methods for the Examination of Water and Wastewater* defines two tests for determining chlorine demand: one for routine laboratory use and the other for field applications.

## Residual Chlorine

The kind and amount of residual chlorine required for a particular application relate to the objectives of treatment and vary from time to time and place to place. Where disinfection is the objective, pH, temperature, chlorine demand, and organism vulnerability or resistivity to chlorine are primary considerations. With long contact periods, sometimes a low concentration of disinfection suffices, whereas short contact times require higher concentrations to accomplish equivalent kills. Moreover, the killing power of chlorine increases with rising temperature, in addition to being markedly influenced by pH. Reference has already been made to the reduction of bactericidal power that occurs with increasing pH; in the case of free available chlorine, it is due to the dissociation of HOCl, and in the case of uncombined available chlorine, to the swing of equilibrium from di- to monochloramine.

The control of chlorination processes is based on the attainment of a definite residual, which, after the contact afforded, has been established by experimentation and experience to accomplish the purposes intended. As already noted, demonstration of bacterial (or other organism) destruction is the only *positive* way of establishing the efficacy of chlorine disinfection. Residual chlorine, manually or automatically determined, is the only *practical* way, however, for relating this with sufficient rapidity to plant operation control, and tests for the determination are universally employed for that purpose. Quality control by bacteriological testing, coupled with statistical interpretations of results, has become largely a matter of confirming treatment efficacy and providing a historical record or of satisfying regulatory requirements.

## Tests for Residual Chlorine

Conventional methods for determining chlorine residuals depend on the oxidizing power of chlorine and are based on reactions with reducing agents. Basically three tests are currently employed: volumetric starch iodide (iodometric titration), modified calorimetric titration with orthotolidine, and amperometric titration.

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**Starch-iodide test.** The starch-iodide test was the only satisfactory basis of chlorination control until about 1913, when the orthotolidine test was developed. It depends on the oxidizing power of free and combined chlorine residuals to convert iodide to free iodine:

$$HCl + HOCl + 2KI \longrightarrow 2\,KCl + I_2 + H_2O \qquad \text{(Eq Z–19)}$$

In the presence of starch, free iodine produces a blue color indicative of the presence of residual chlorine; the depth of color indicates the chlorine content. Quantitative measure of *total* residual chlorine (other oxidants react similarly) is achieved by acid titration of released iodine with a standard reducing agent solution, commonly sodium thiosulfate, using starch as the indicator:

$$I_2 + 2Na_2S_2O_3 \longrightarrow Na_2S_4O_6 + 2NaI \qquad \text{(Eq Z–20)}$$

Although a familiar test to all water utility analysts, it is not always satisfactory as a means of treatment control because when the chlorine concentration is less than 1 mg/l the titration end point is difficult to detect. Moreover, its utility is limited by the lack of specificity to chlorine as compared with other oxidants. Its principal applications are for calibration of stock chlorine solutions where total chlorine concentrations exceed about 10 mg/l, for preparation of chlorine water used in making temporary color standards and for determining total residual chlorine concentration in excess of 0.04 mg/l.

**Basic orthotolidine (OT) test.** Orthotolidine is an aromatic compound that is oxidized in acid solution by chlorine, chloramines, and other oxidants to produce a yellow-colored complex, the depth of which, at pH levels below 1.3, is proportional to the amount of chlorine and other oxidants present. The test is especially suitable for routine determination of *total* chlorine residuals of 0.01 mg/l but not exceeding about 10 mg/l. The calorimetric OT test is sensitive, fairly rapid (about 5 min reaction time is required), and simple, but it is somewhat affected by natural color and turbidity, temperature variations, nitrites, ferric and, especially, manganic ions, and possibly by organic iron, algae, and other materials. Because of these inherent limitations and the difficulties in compensating for many of them, some investigators believe that the basic test is unsuitable for research applications and for continuous automatic recording of chlorine residuals. The test is widely used for routine measurement of residual chlorine in plant control and in the field.

An orthotolidine flash test modification, performed at a temperature near 1C to minimize the effect of chloramines and their reaction with orthotolidine to produce a yellow color, can be used in the qualitative determination of *free* available chlorine in excess of 0.3 mg/l. Although oxidized manganese affects the test results, slow-acting interfering substances, nitrites, and oxidized iron do not have such a significant influence. The flash test has the disadvantage of requiring the reading of color (and matching with color standards) within five sec. It is an exceedingly useful and simple control procedure where it is necessary to maintain a definite amount of free available chlorine.

With the development of current concepts of free and combined available residual chlorination, test methods have been designed to permit better differ-

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entiation and quantitative evaluation of residual chlorine forms. The following two test methods are commonly employed for this purpose.

**Orthotolidine-arsenite (OTA) test.** The unique characteristics of sodium arsenite as a reducing agent led to the development of the OTA test for quantitative determination and differentiation of *free and combined* available chlorine forms. The test is based, in part, on the fact that free chlorine residuals react instantaneously with orthotolidine whereas combined residuals react slowly. *Total* chlorine is first determined by the OT methods (as previously discussed), with reading made after 5 min. Another test is made on a fresh sample, adding sodium arsenite reducing agent immediately (within 5 set) after the orthotolidine; this permits oxidation of orthotolidine by free chlorine, but allows only a small part of combined chlorine to react before the addition of arsenite (which reduces the chloramines instantly, preventing further reaction with orthotolidine) and subsequent color development. The resulting color intensity of this sample thus is primarily a measure of free chlorine. If interferences are present, additional test modifications are indicated to correct the values of total and free chlorine; the addition of sodium arsenite before the orthotolidine neutralizes all chlorine compounds in the water, and any color formed is due to the reaction of interfering substances with the orthotolidine.

Both the OT and OTA test methods can be rapidly completed by a simple visual apparatus or by photometric techniques. Various residual chlorine test kits employing either permanent glass or liquid OT color standards are available and widely used. Most units are designed to compensate for the effects of color and turbidity.

**Amperometric titration.** The most accurate and complete test for determining total residual chlorine or for differentiating free and combined forms involves the use of an amperometric titrator with specific reagents and within specific pH ranges in sequential steps similar to those discussed. The titrator is used to determine the end point of a titration that is accomplished through the addition of a reducing agent. Current flow is proportional to the oxidant concentration and decreases as the oxidation-reduction reactions progress. When these actions are complete, the current flow will remain quite constant (the end point of the titration) and will not be decreased by further addition of the reducing agent.

A standard solution of phenylarsenoxide ($C_6H_5AsO$), the reducing agent normally used in the titration, quantitatively reduces free chlorine (and, proportionately, current flow) at a pH between 6.0 and 7.5, but does not react with combined chlorine in this pH range. Phenylarsenoxide quantitatively reduces combined chlorine in the presence of potassium iodide in the pH range of 3.5 to 4.5. Accurate determination of free chlorine cannot be made in the presence of nitrogen trichloride, chlorine dioxide, other halogens, copper, and silver, but other substances that interfere with the OT test (such as manganese, nitrite, and iron) do not affect the amperometric test. This test method is, of course, unsuitable for field use, and it involves a greater degree of skill and care than those methods described previously.

An adaptation of the amperometric technique is available for differentiation of monochloramine and dichloramine. The differentiation depends on the fact that monochloramine reacts more readily with iodide than does dichloramine. Different iodide concentrations and pH values are used for separate determinations. The adaptation is subject to the usual interfering influences; in

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addition, nitrogen trichloride, if present, titrates partly as free chlorine and partly as dichloramine, and it cannot be determined in which fraction any organic chloramines that might be present will titrate.

**Other tests.** Other test methods (such as those developed and refined by Palin) for differentiating available chlorine forms involve serial titration using orthotolidine as the indicator. At pH 6.3-6.5 and 20C, free chlorine, nitrogen trichloride, and chlorine dioxide form a blue color with orthotolidine that can be titrated to decolorization by standard ferrous ammonium sulfate solution. By adding a trace of potassium iodide, monochloramine concentration can be similarly determined (in addition to some organic chloramines). Dichloramine concentration can be determined in a third step by adding sulfuric acid and more iodide, buffering with sodium carbonate to pH 6.5, and titrating again with the ferrous ammonium sulfate solution. Interferences due to chlorine dioxide, trichloramine, and manganese can be determined, but organic chloramines, as in the amperometric titration technique, titrate as either monochloramine or dichloramine. The complexity, time, and skill required in the performance of the complete differentiation currently limit application of the technique to research and specialized plant control problems.

In Palin's N, N-diethyl-p-phenylene Diamine (DPD) monohydrochloride test method, reagents in granular or compressed tablet form can be used. With free chlorine the indicator produces a red color. Like the Palin method using orthotolidine as the indicator, free chlorine, monochloramine, and dichloramine can be determined, and interferences can be suppressed by using appropriate chelating agents.

The determination of chlorine dioxide is quite simple when this material is in solution by itself, and any of the techniques for chlorine may be adapted for the purpose. When present with hypochlorous acid, its determination is more difficult. At concentrations in excess of 0.2 mg/l, it can be determined specifically with tryosine, but this calorimetric procedure is not considered sufficiently sensitive for most water-treatment-plant needs. The determination of chlorine dioxide in the presence of chlorine can be accomplished with the usual techniques for measuring chlorine after the latter has been preferentially reduced by oxalic or malonic acid.

## Automatic Residual Analyzers and Recorders

Where variations in chlorine demand are erratic and fluctuate between wide limits, a continuous indication and record of residual chlorine is especially desirable. Equipment for this purpose is an essential part of automatic chlorination instrumentation, and can be installed in any convenient location at or remote from the sampling cells.

Where residual chlorine recording facilities are provided, more effective and economical chlorination practices are assured. Residual and recording systems are especially useful in detecting and compensating for sudden or natural changes in chlorine dosage, and furnish a permanent record of the efficiency of chlorine treatment. Remote location of recorders facilitates efficient and prompt adjustments of chlorine feed from central operating stations and minimizes the necessity for continuous, on-the-spot surveillance of treatment facilities. The operational principles of residual analyzers are discussed at length in Chap. 8.

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CHAPTER 7

# Chlorination—Safe Handling Practices

The physical, chemical, and toxicological characteristics of chlorine and other chemicals used in connection with water disinfection have been detailed elsewhere. Many water chlorination chemicals, notably the compressed gases, require special care in handling and are thus subject to strict governmental controls. Additionally a number of recommendations and practices have been developed to ensure maximum safety for those associated with the use of these gases and for others who might be affected by their accidental release.

## Types of Shipping Containers for Chlorine Compressed Gases

Classified by the US Department of Transportation and the US Coast Guard as nonflammable compressed gases, chlorine (as well as ammonia and sulfur dioxide) must be packaged in containers that comply with DOT and/or Coast Guard Regulations regarding loading, handling, and labeling when shipped in the US by rail, water, or highway. State, local, and insurance requirements must also be adhered to. DOT regulations limit the maximum filling capacity of all compressed gas containers (i.e., the per cent ratio of the weight of the gas in the container to the weight of water that the container will hold); for chlorine containers this limit is 125 per cent (approximately 88 per cent of total container volume). A variety of containers are available to meet individual customer needs; all are steel, are often equipped with one or more safety devices, and are pressure-tested at regular specified intervals.

**Cylinders.** Cylinders are seamless bottles hot-drawn from one piece of steel. The most popular cylinders for water utility practice have a commodity capacity of 100 or 150 lb, but many other sizes are available.

Regulations limit the maximum filling of chlorine cylinders to 150 lb and permit only one opening in the cylinder — the valve connection at the top. All chlorine cylinders must contain a fusible metal safety device designed to melt at between 158F and 165F. It is an integral part of the standard operating valve. One type has a fusible metal cast directly into a threaded hole in the valve body,



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and another has a threaded bronze plug containing the fusible metal, which is screwed into a tapped hole in the valve body. This safety device releases the container contents and prevents hydrostatic failure from excessive pressure attendant on exposure of the container to fire or other excessive heat. Chlorine cylinders filled to the maximum legal limit become completely liquid full at about 154F. Cylinders of 100-lb capacity vary in diameter from 8 1/4 to 10 1/4 in., in height from about 40 to 59 in., and have an empty weight from about 63 lb to 115 lb. Cylinders of 150-lb capacity vary in diameter from 10 1/4 to 10 3/4 in., in height from about 53 to 56 in., and have an empty weight from about 85 to 140 lb.

Regulations permit shipment of chlorine cylinders by rail in either carload or less-than-carload lots. Shipments by motor vehicles are also permitted in less-than-carload lots or in truck-load quantities of various minimum weights, depending upon the motor carrier and the area of transportation.

**Ton containers.** Ton containers for chlorine are welded, cylindrical steel tanks, usually having convex-inward heads and sides crimped inward to form chimes that provide a substantial grip for lifting them. (Some older containers have compound curvature of one end.) Contemporary containers are usually equipped with six fusible metal plugs, three in each end located 120 deg. apart. They also are equipped with two operating valves at one end, each of which is connected inside the container to an education pipe terminating near the side wall. These containers, having a diameter of about 30 in. and an average length of about 80 in., vary in empty weight from 1,300 to 1,600 lb. They are filled to about 2,000 lb of chlorine.

Chlorine ton containers are authorized for railroad shipment but only on a special tank car frame known as a multiunit tank car. This car (TMU) is designed to hold fifteen containers. The initial rail shipment of these containers is unloaded from the car at the consumer's plant. In subsequent shipments the full containers are exchanged for empties returned for refilling. (No freight is charged on the return carload of empty containers since tank cars are entitled to free return movement.) It is impractical to ship less than fifteen containers because the rail transportation fee is figured for fifteen full containers at prevailing carload rates. TMU cars must be consigned for delivery and unloading on a private track; where a *private* track is not available, the containers may be removed from the car frame on carrier tracks if proper written permission is first obtained. Regulations also provide that one or more ton containers may be transported on trucks or semitrailers when securely chocked or clamped, and provided adequate transfer facilities are available when container transfer in transit is necessary.

**Other shipping containers.** DOT regulations permit shipment of compressed gases in a variety of other containers, such as tank cars (rail), tank trucks, and barges.

Single-unit chlorine tank cars, which by regulation, must be unloaded on a private track, are available in 16-, 30-, 55-, 80-, and 90-ton capacities. These containers, unlike cylinders and ton containers, are designed to be unloaded as *liquid,* which must be vaporized to gas where process requirements so indicate; in a few instances, they are unloaded into stationary storage tanks. Large

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consumers along waterways might choose to consider barge chlorine shipments; those consumers having no private track facilities and not immediately adjacent to suitable waterways might choose to consider tank truck shipments. In both cases, if such shipments can be provided, transfer of container contents to stationary storage tanks is necessary.

### Handing Shipping Containers

Various mechanical devices, such as skids, troughs, and up-ending cradles, facilitate handling of chlorine cylinders. When unloading from trucks or platforms, cylinders must not be dropped to ground level. If cylinders must be lifted and an elevator is not available, specially designed cradles or carrying platforms in combination with a crane or derrick are recommended; chains, lifting magnets, and rope slings that encircle the cylinders are unsafe and should not be used. For lateral movements a properly balanced handtruck is useful. Cylinders being moved should always have valve protection hoods in place; these hoods are not designed to hold the weight of cylinders and their contents, and cylinders should never be lifted by this means.

Ton containers may be moved by various methods, such as by specially fitted trucks and dollies, a monorail system, or rolling. Where it is necessary to lift them, as from a TMU car or truck, a suitable lift clamp in combination with a hoist or crane of at least 2-ton capacity should be used. Valve protection hoods should always be in place when ton containers are being moved.

Receiving and unloading areas and safety precautions applicable to handling single-unit railroad tank cars, tank trucks, and other shipping containers are subject to strict regulations.

### Storing Shipping Containers

Chlorine cylinders preferably should be stored upright and secured, and in such a manner as to permit ready access and removal. Ton containers should be stored horizontally, slightly elevated from ground or floor level, and blocked to prevent rolling; a convenient storage rack is obtained by supporting both ends of containers on rails of I-beams. Ton containers should not be stacked or racked more than one high unless special provision is made for easy access and removal. Full and empty cylinders and ton containers should be segregated.

Storage areas should be clean, cool, well ventilated, and protected from corrosive vapors and continuous dampness. Cylinders and ton containers stored indoors should be in a fire-resistant building, away from heat sources (such as radiators, steam pipes, etc.), flammable substances, and other compressed gases. Subsurface storage areas should be avoided, especially for chlorine and sulfur dioxide. If natural ventilation is inadequate, storage and use areas should be equipped with suitable mechanical ventilators. Cylinders and ton containers stored outdoors should be shielded from direct sunlight and protected from accumulations of rain, ice, and snow.

All storage, handling, and use areas should. be of such design that personnel can quickly escape in emergencies. It is generally desirable to provide at least two means of exit. Doors should open out and lead to outside galleries or platforms, fire escapes, or other unobstructed areas.

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## Emptying Containers

Chlorine cylinders deliver gas when in an upright position and liquid when in an inverted (or partially so) position. Ton containers in a horizontal position and with the two valves in a vertical line deliver gas from the upper valve and liquid from the lower valve.

To withdraw gas from a cylinder or ton container, the liquid form of the material must be vaporized. The flow rate is a function of the vaporization rate, which, in turn, is dependent on the rate of heat transfer to the liquid.

The dependable, continuous discharge rate of chlorine gas from a 100-lb or 150-lb cylinder, at about 70F under normal circulation conditions and without sweating, is about 1 3/4 lb/hr against a 35-psi back pressure. Withdrawal rates of 4 lb/hr can be obtained against a negative or zero back pressure from a 150-lb cylinder and 20 lb/hr from a ton container. Under similar conditions, the gas discharge rate from a ton container is about 15 lb/hr. If sweating can be tolerated, these rates can be doubled; for short periods they may be greatly exceeded. Heat must never be applied to any type of chlorine container in an effort to increase the vaporization and, hence, discharge rates. If the gas discharge rate from a single chlorine cylinder or ton container is inadequate, two or more can be connected to a manifold and discharged simultaneously. A scale should be provided since this is the only accurate means of determining container contents. The dependable, continuous discharge rate of liquid chlorine, at about 70F and against a 35-psi back pressure, is about 200 lb/hr for cylinders and 400 lb/hr for ton containers. Batch discharge rates may be considerably higher. Connection of cylinders or ton containers discharging liquid chlorine to a manifold is not recommended.

Emptying of single-unit chlorine tank cars should be accomplished through a suitable metal flexible connection to accommodate the rise of the car as its springs decompress. These cars almost invariably are emptied by discharging liquid. The liquid chlorine may be unloaded by its own vapor pressure. Cold weather usually decreases the unloading rate; in extremely cold climates it may be helpful to unload cars in a shed maintained at about 70F. If necessary, the car pressure can be increased by padding with clean dry air, sometimes done by the tank-car loader, and occasionally by the consumer. Vaporizers must be used when the consuming process requires gas.

In general, requirements for emptying chlorine tank trucks and barges are similar to those of tank cars. Suitable chlorine bulk-storage facilities are also required. In the case of barges, piping, loading, and unloading arrangements must be approved in the US by the Coast Guard. Where bulk storage is indicated (e.g., when chlorine is shipped by barge or, in most cases, by tank truck), a sufficient pressure differential must be maintained between the shipping container and the storage containers. Facilities must be provided to vent chlorine gas or chlorine-air mixtures from the storage tank to the consuming process or other disposal system; venting may be required during the entire unloading period. When transferring chlorine from the storage tank to the consuming process, air padding may be necessary. The procedure is essentially

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the same as that required for emptying tank cars. Reliable means must be provided for determining tank contents; weight-measuring devices are preferred, and gage glasses should not be used. Especially where night operations are contemplated, adequate lighting, including auxiliary sources, is essential. Storage tanks should be of such number and capacity as is consistent with the size of shipments received and rate of consumption. A thorough investigation of all pertinent regulations and insurance restrictions regarding their design, capacity, and location should be made prior to beginning construction.

### Connecting and Disconnecting Containers

The design and operation of facilities should be such as to minimize all hazards associated with connecting, emptying, and disconnecting chlorine containers. These operations should be performed in well-lighted places by authorized personnel equipped with gas masks or other suitable respiratory protection devices. Container-valve protection hoods should always be in place when the container is not in use. Valves should not be left open when operating personnel are not available to maintain proper surveillance of the operations.

Connections to valve outlets on cylinders and ton containers can be made by either a clamp and adapter or a union connector; the former is preferred. In making connections it should be ascertained that the outlet valve is closed before the outlet cap is removed. Gasket surfaces should be thoroughly inspected and cleaned and a new gasket of standard material should be used. Connections that do not fit should never be forced.

Cylinder and ton container valves should be slowly opened by using a special wrench, no more than 6 in. long, for this purpose. One complete turn of the stem in a counterclockwise direction opens the valve sufficiently to permit maximum discharge. An auxiliary cylinder or ton container valve should be installed adjacent to the container valve between it and the chlorine feeder, or gas header on manifolded systems. Such a valve serves as an emergency shutoff if the container valve should leak. Moreover, it prevents chlorine gas from escaping from the supply line when the container is removed from service. In the interests of safety, the ventilation system should be operating whenever containers are being placed into or removed from service and at all times in which an emergency exists or adjustments and repairs are being made.

Tank-car angle valves should be connected and disconnected using a pipe wrench no longer than 18 in. In discontinuing the flow of liquid chlorine from tank cars (and ton containers), the supply line to the evaporator should be emptied since expansion of any remaining chlorine may build‘ up hydrostatic pressure and burst the line. The tank-car angle valve (or lower ton-container valve) should be closed first and the line evacuated by continuing the use of any chlorine remaining before the evaporator inlet valve is closed.

All containers should be tested for leaks when being placed into or removed from service. If a leak is evident around the valve stem and all other precautions have been observed, it usually can be eliminated by tightening the packing nut. Chlorine leaks always get worse; help should be solicited if they are serious and they cannot readily be eliminated.

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## Engineering Control of Hazards

Careful consideration should be given to methods of handling chlorine shipping containers. Ceilings high enough for overhead hoists and floors of sufficient construction for ease in handling mechanical equipment should be provided. Storage and use areas should be properly ventilated so as to minimize possible accidental reaction of chemicals, which is based on their previously noted characteristics. Piping systems should be as simple as possible with a minimum number of joints; they should be well supported, protected against temperature extremes, and adequately sloped to allow proper drainage.

Long pipelines for liquid chlorine should be avoided. Sections of pipelines that can be isolated or shut off at both ends (such as by valves) must be provided with a suitable expansion chamber to avoid possible hydrostatic rupture due to pressure and volume increases accompanying high temperatures (Fig. 2). Condensation or reliquefaction can occur in chlorine gas lines that pass through areas where the temperature is below the temperature-pressure equilibrium indicated on the chlorine vapor pressure curve (Fig. 1). It can be prevented by supplying properly controlled heat or by reducing the pressure. The temperature of chlorine containers and gas pipelines should be lower than the temperature of the chlorinator to prevent condensation; if chlorine gas condenses, the chlorine will be reliquefied and result in erratic chlorinator operation. Chlorine pipelines removed from service for even very brief periods should be closed in a suitable manner to preclude the entrance of moisture, which will cause serious corrosion problems.

Equipment cleaning and repairs should be performed under the direction of thoroughly trained personnel who are fully familiar with all of the hazards and the safeguards necessary for the safe performance of the work. All precautions pertaining to education, protective equipment, and health and fire hazards should be reviewed and understood. Repairs to chlorine systems should not be undertaken while the system is in operation and while piping systems are in service. Chlorine pipelines and equipment should be first purged with *dry* air as a safeguard to health; this is especially important where cutting or welding operations are undertaken because iron and steel will ignite in chlorine at about 450F. Immediate drying of a chlorine pipeline into which water or moisture has been introduced, or which has been opened for repairs, is essential if corrosion is to be prevented.

The technical problems of designing storage and use areas, equipment, and piping systems; providing adequate ventilation; and formulating operating procedures that insure maximum security and economy can best be handled by experienced engineers and safety and fire protection specialists. Assistance and literature are available from such organizations as The Chlorine Institute, Inc., the Compressed Gas Association, and the Manufacturing Chemists' Association, and from material and equipment producers and suppliers.

Some additional precautions, recommendations, and safety practices regarding handling and use of hazardous chlorination chemicals are outlined in the AWWA's "Safety Practice for Water Utilities" and the Water Pollution Control Federation "Safety in Wastewater Works" manuals.

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## Finding Gas Leaks

The characteristic odor of chlorine provides warning of leaks. Chlorine reacts with ammonia to form dense white fumes; a supply of strong ammonia water (commercial 26° Bé, as household ammonia is not sufficiently strong) should be available for locating leaks. The leak can be readily detected by holding an ammonia-saturated cloth near the suspected area. Compressed gas containers (in storage and use), piping, and equipment should be tested for leaks daily.

## Handing Emergencies

As soon as there is indication of a chlorine leak or other abnormal condition, corrective steps should be taken. Leaks never get better by themselves; they *always* get worse if not promptly and suitably repaired. Authorized, trained personnel with suitable gas masks should investigate, and all other persons should be kept away from the affected area. The ventilation system should be placed in operation immediately. Unconfined chlorine, being heavier than air, tends to lie close to ground levels. (This characteristic must be kept in mind in designing chemical storage and use areas and appropriate natural or mechanical ventilation systems.) If leaks cannot be handled promptly, the chemical supplier or nearest office or plant of the producer should be called immediately for emergency assistance.

In case of fire, containers should be removed from the fire zone immediately. Portable tanks, tank cars, trucks, and barges should be disconnected and, if possible, also removed from the fire zone. *If there are no leaks*, water should be applied to cool the container. Water must never be applied to a *leaking* chlorine container. Chlorine is only slightly soluble in water and the corrosive character of its reaction with water *always* will intensify the leak. In addition, the heat supplied by even cold water will increase the vaporization rate. Leaking chlorine containers similarly should not be thrown into a body of water because the leak will be aggravated and the container might float when still partially full, allowing uncontrolled gas evolution at the surface.

If a leak occurs in equipment or piping, the supply should be discontinued and the material under pressure at the leak should be disposed of. Leaks around container valve stems usually can be stopped by tightening the pack nut or gland. If this action does not stop the leak, the container valve should be closed and material under pressure in the outlet piping should be disposed of. If the valve does not shut off tight, the outlet plug or cap should be applied. In the case of a leaking valve of a ton container, the container should be positioned so that the valves are in a vertical plane with the leaky valve on top.

If one is confronted with other container leaks, one or more of the following procedures should be considered:

1. Position cylinders or ton containers so that gas instead of liquid escapes. The containers may be insulated with sacks, earth, etc., to decrease absorption of heat and the discharge rate.
2. Apply appropriate emergency capping devices, if available.
3. Call the supplier or nearest producer for emergency assistance.
4. If practical, reduce pressure in the container by removing the gas to

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process or suitable disposal system. Caustic soda, soda ash, or other suitable alkali absorption system should be provided for disposing of chlorine from leaking cylinders and ton containers (100 lb of $C1_2$ can be neutralized with 125 lb of caustic soda).

5. In some cases it might be desirable and possible to move the container to an isolated spot where it will do the least harm.

## Personnel Protective Equipment

Severe exposure and potential health hazards exist wherever noxious compressed gas and other respiratory irritants are handled or used. An approved gas mask should be provided for every employee involved with handling them. Additionally, suitable protective equipment for emergency use should be available outside rooms where hazardous materials are located, near the entrance, away from areas of likely contamination. Such equipment might be provided in several locations at larger installations. Only respiratory protective equipment approved by the US Bureau of Mines specifically for chlorine service may be used.

Canister-type gas masks with a full face piece and specific or all-purpose commodity canister should be used only for relatively short exposure periods, and only if it has been clearly established that sufficient oxygen is present (not less than 16 per cent in air) and that contamination does not exceed the allowable level (1 per cent for chlorine). Canister masks might not be useful in emergencies since these criteria might not be readily ascertained, especially if suitable forced-ventilation schemes are not provided. Regular replacement of over-age canisters, even though unused, is recommended.

Self-contained breathing apparatus, with a full face piece and a cylinder of air or oxygen carried on the body, or with a canister that produces oxygen chemically, is suitable for high contaminant concentrations and is the preferred means of respiratory protection. Protection is provided for a period that varies with the amount of air, oxygen, or oxygen-producing chemicals carried.

Positive pressure (blower) hose masks, compressed-air line masks, or combination compressed-air line masks and self-contained breathing apparatus are also suitable, provided the masks and accessories have been approved by the NIOSH. The air supply must be free of dust, oil, and carbon monoxide.

Respiratory protective equipment should be carefully maintained, inspected, and cleaned after each use and at regular intervals. Defective or inoperable equipment is worse than none at all. All such equipment should be used and maintained in strict accord with the manufacturer's instructions. No person should enter contaminated areas unless attended by an observer who can rescue him in event of respirator failure or other emergencies.

It is good practice to provide eye-protection devices (or masks with full face pieces) and other protective clothing for workers exposed to hazardous materials. Emergency showers, eye baths, or other suitable water-flush systems should be provided in convenient locations for use by accidentally exposed personnel. Installation of an automatic chlorine leak detector, with or without visible or audible alarm, should be considered.

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## Personnel Training

Safety in handling hazardous materials depends, to a great extent, upon the effectiveness of employee education, proper safety instructions, intelligent supervision, and the use of safe equipment. Training for both new and old employees should be conducted periodically to maintain a high degree of safety in handling procedures. Employees should be thoroughly, informed of the hazards that may result from improper handling. They should be cautioned to prevent leaks and thoroughly instructed regarding proper action to take in case leaks do occur. Each employee should know what to do in an emergency and should be fully informed as to first-aid measures.

In addition, employee training should encompass the following:

1. Instruction and periodic drill or quiz regarding the locations, purpose, and use of emergency fire-fighting equipment, alarms, and emergency crash shut-down equipment, such as valves and switches
2. Instruction and periodic drill or quiz regarding the locations, purpose, and use of personal protective equipment
3. Instruction and periodic drill or quiz regarding the locations, purpose, and use of safety showers, eye baths, bubbler drinking fountains, and the closest source of water for use in emergencies
4. Instruction and periodic drill or quiz of selected employees regarding the locations, purpose, and use of respiratory first-aid equipment
5. Instruction to avoid inhalation of toxic vapors and all direct contact with corrosive liquids
6. Instruction to report to the proper authority all leaks and equipment failures

## Special Care and Usage of Hypochbrites

High-test calcium hypochlorite is relatively stable under normal atmospheric conditions. However, reactions may occur spontaneously with organic materials such as turpentine, ether, oils, sugar, fats, and other oxidizable substances. Hypochlorites should be stored and handled, therefore, using only clean dry implements free of oils, grease, and other organic impurities.

Because of its strong oxidizing powers and reactivity, calcium hypochlorite, especially in large quantity, preferably should be segregated or stored in a location separate from other chemicals or materials with which it might react. For example, chlorine released from a leaking hypochlorite container in the proximity of activated carbon might be absorbed so rapidly that sufficient heat would be generated to cause the carbon to reach the ignition temperature and take fire.

Whenever a leaking or defective container is noted, it should be immediately removed from the storage area as it can rapidly cause corrosion of other containers and equipment in the area. Moisture, in addition to other factors mentioned, contributes to hypochlorite deterioration and indicates the desirability of storage in closed containers of such size that the contents will be consumed in a relatively short period, depending on process requirements.

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Solutions of calcium hypochlorite are corrosive; thus, they should be handled in corrosive-resistant materials such as plastic, ceramic, glass, wood, or in rubber-lined steel vessels. For liquid feeding of high-test calcium hypochlorite, solutions of about 1 or 2 per cent available chlorine content are usually recommended. Do not mix calcium hypochlorite with anything except water.

The corrosive nature of sodium hypochlorite also dictates the use of corrosion-resistant material for handling and dispensing, and the use of protective equipment by persons involved in these operations. Accordingly, rubber gloves and aprons, goggles, or other suitable protective apparel should be worn when one is preparing and handling hypochlorite solutions. In addition, every precaution should be observed to minimize splashing from the container. Accidental splashes should be treated immediately by removing affected clothing and washing it and all exposed parts with copious quantities of water.

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CHAPTER 8

# Chlorination Equipment

## Chlorinators

**Types of feeders.** Chlorine gas feeders may be classified into two types.

1. *Direct-feed type.* Chlorine gas under pressure is delivered directly to the point of application. (This system is sometimes referred to as dry feed or direct feed .)

2. *Solution-feed vacuum type.* Chlorine gas is maintained under vacuum throughout the apparatus. An injector is employed into which the gas is drawn to provide solution feed and create the vacuum. In this feeder, the vacuum governs the operation of the chlorine inlet valve directly or indirectly so that the chlorine will not feed unless sufficient vacuum is induced through the apparatus. This type of feeder is most common because safe operation is assured. It employs a direct indicating meter, and the flow of chlorine is automatically shut off on loss of vacuum, stoppage of discharge line, or loss of operating water pressure.

**How the feeder operates.** Basically, a chlorine gas feeder is a simple apparatus. It operates on the principle of regulating the flow of chlorine gas by controlling conditions existing upstream and downstream of an orifice in a flow line. This orifice may be constant or variable. Various additional refinements are incorporated in the modern chlorine gas feeder, but they do not alter the basic principle. The refinements are present to assure low maintenance and long life and to afford greatest possible protection to operating personnel and to the premises in which the feeder is located.

At the top of Fig. 15 is a diagram representing the flow of any fluid through a pipeline. The rate of flow may be determined by introducing into the line an orifice of known size in order to create a differential-pressure condition. With no flow through the line, the pressure indicated on the piezometer tubes, upstream and downstream of the orifice plate, will be the same. As flow takes place, a pressure drop occurs across the orifice and is indicated by the difference in height of the fluid in the piezometer tubes.



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The difference in pressure, commonly referred to as "differential," is definitely related to flow for any given size of orifice. Therefore, for a known size of orifice, a measure of the pressure differential is a measure of the rate of flow. It follows that if means are provided whereby given differentials may be set up, known flows may be established through a line in which an orifice of a selected size is inserted. This basic principle may be applied to create a device used to establish and control chlorine gas feed according to the will of the operator.

**Method A — upstream pressure variable.** The manner in which this principle is applied can be understood by referring to the diagram in Fig. 15 (Method A). To simplify the rate-setting operation, the vacuum downstream of the orifice is maintained constant. That is, the chlorine feed apparatus is so designed that regardless of the rate of chlorine feed through the system, no change takes place in the downstream vacuum. This condition, maintained under vacuum for convenience and for safety, is represented as *Vc.* Since the orifice is of a constant selected size and since one "leg" of the differential is maintained constant, the other "leg" is the factor that determines the rate of feed.

For every pressure established upstream of the constant orifice (downstream vacuum is constant), chlorine will feed at a given rate through the system. The

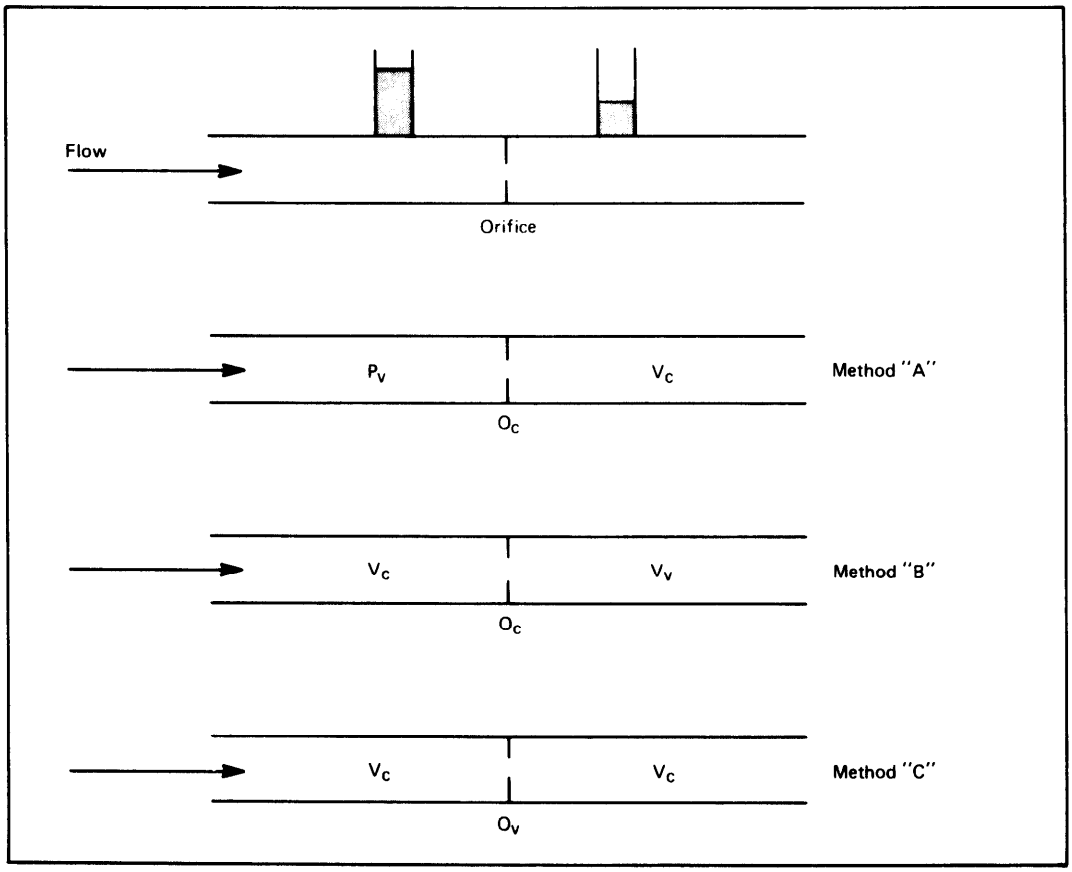


**Fig. 15.** Fluid Flow Through a Pipe Line

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upstream condition is represented as *Pv,* indicating that this is a pressure condition that may be varied at the will of the operator.

**Method B — downstream vacuum variable.** In this case (Method B), the orifice is maintained constant and a constant condition is established upstream of it. Rate of feed is varied, then, by changing the vacuum that exists downstream of the orifice — still, in effect, creating a known differential across the orifice. As in Method A, two conditions are kept constant: the orifice and one of the differential legs. Feed can be governed, obviously, by. maintaining any two conditions constant and varying the third.

**Method C — orifice variable.** In a third method of accomplishing control (Method C), the differential is maintained constant, and the rate of flow is governed by changing the size of the orifice. This method might be considered somewhat analogous to a faucet discharging to atmosphere with a constant supply pressure behind it. The amount of water discharged can be governed by gradually opening the faucet, and as long as the supply pressure does not change, the rate of flow will be maintained constant for any given setting.

Basically, therefore, a chlorine feeder utilizes an orifice of some type in a gas flow line with means for keeping certain conditions constant and with means for establishing at will other conditions and maintaining them at this established value. All other devices incorporated in chlorine feeders are included for convenience and safety.

**The complete chlorine feeder.** By referring to Fig. 16, with the basic operating principle in mind, it is possible for one to follow the evolution of a chlorine feeder from its simplest form to one that is "complete."

Figure 16, line 1, shows the simplest type of chlorine gas feeder. It consists of a length of silver tubing attached to a chlorine cylinder. The tank valve is employed as an adjustable orifice. The cylinder maintains a pressure condition upstream of the valve, and the gas is discharged to a point of application under a reasonably constant head. In effect, therefore, conditions upstream and downstream of the orifice are maintained reasonably constant, and rate of feed can be varied by varying the size of the orifice.

*Note:* Lines 1, 2, and 3 of Fig. 16 are used to illustrate the evolution of modern chlorinators. They are not to be used to construct actual feeding devices except in extreme emergency.)

**Check valve.** To protect the apparatus, a check valve is added, as shown in Fig. 16, line 2. The check valve prevents water from being drawn back; it is the first refinement incorporated to protect the apparatus and personnel and to assure satisfactory operation.

To avoid the necessity for frequent residual tests to guide the operator in establishing proper feed rate, the next logical step in the evolution of a chlorine feeder is to add a metering device, as shown in Fig. 16, line 3. The flow meter, a visible-flow indicator, is a direct-reading variable-area type through which all of the chlorine flows. It will show zero flow when no gas is being fed and will accurately indicate any variations from set feed rate. Changes in flow rate can be made by adjusting the variable orifice *(Ov)* placed after the indicator.

A meter having been established, the next logical step is to strive for dependable maintenance of flow rate. Since chlorine cylinder pressure varies

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with temperature, the condition upstream of the throttling valve will not remain constant. The condition downstream also may vary because of changes in conditions of head at the point of application. These variable conditions necessitate frequent adjustments of the valve setting to maintain an approximately constant rate of feed.

**Addition of compensating valves.** To assure that these variable conditions will not affect control, compensating valves are added upstream and downstream of the orifice. The design of these valves will be dictated by the control method used (Fig. 15). Figure 16, line 4, shows the arrangement if the conditions downstream and upstream are to be kept constant and the orifice variable to control flow. The outlet-regulating valve is designed to maintain a constant condition downstream of orifice *Ov.* The inlet-regulating valve is designed to maintain a constant condition upstream of the orifice. With controlled conditions upstream and downstream of the orifice, any change in the orifice is reflected in chlorine feed changes.

The pressure-type, direct-feed device described in the foregoing will accurately control the flow of chlorine introduced directly into a pipe or channel carrying the flow to be treated. This device has certain limitations. It is not foolproof, inasmuch as the gas is under pressure throughout the system; any piping leak will allow gas to escape. Chlorine feeders of this type are available and in limited use for special applications.

To overcome the difficulty in attempting to introduce chlorine gas directly at the point of application, another device is added to bring the chlorine gas into intimate contact with a supply of water and thus produce a chlorine solution. This solution is concentrated and relatively stable; it can be introduced safely and easily into an open channel, a closed pipeline, or the suction end of a pump. An injector, sometimes called an aspirator or eductor, is an excellent mixing device. The application of an injector to a direct-type feeder is shown in Fig. 16, line 5. The injector also will induce a vacuum throughout the piping system of the feeder. This effect overcomes one of the weaknesses of the direct-type feeder, in that gas cannot escape while the unit is operating, even if a slight leak occurs in the piping.

**Provision of pressure relief.** A pressure-relief valve in the upstream section is held closed by vacuum. If vacuum is lost and chlorine under pressure passes the inlet valve, the relief valve opens and chlorine gas vents to outside of building. This valve is shown in Fig. 16, line 6.

In summary, with all these refinements (Fig. 16, line 6) a device is created that provides: (1) means for establishing and controlling conditions to feed chlorine accurately; (2) a vacuum-governed system that will prevent gas leaks and automatically shut off chlorine gas under any operating malfunction; (3) an injector to create the operating vacuum and a solution feed for safe and efficient application of chlorine to flow being treated; (4) an indicating device to show rate of feed; (5) a convenient means for setting or controlling the rate of feed; (6) a pressure-relief safety device to vent gas to outside atmosphere if the gas shutoff valve should leak during shutdown; (7) a check valve to prevent backflow during shutdown. Added accessories include chlorine gas pressure and vacuum gages.

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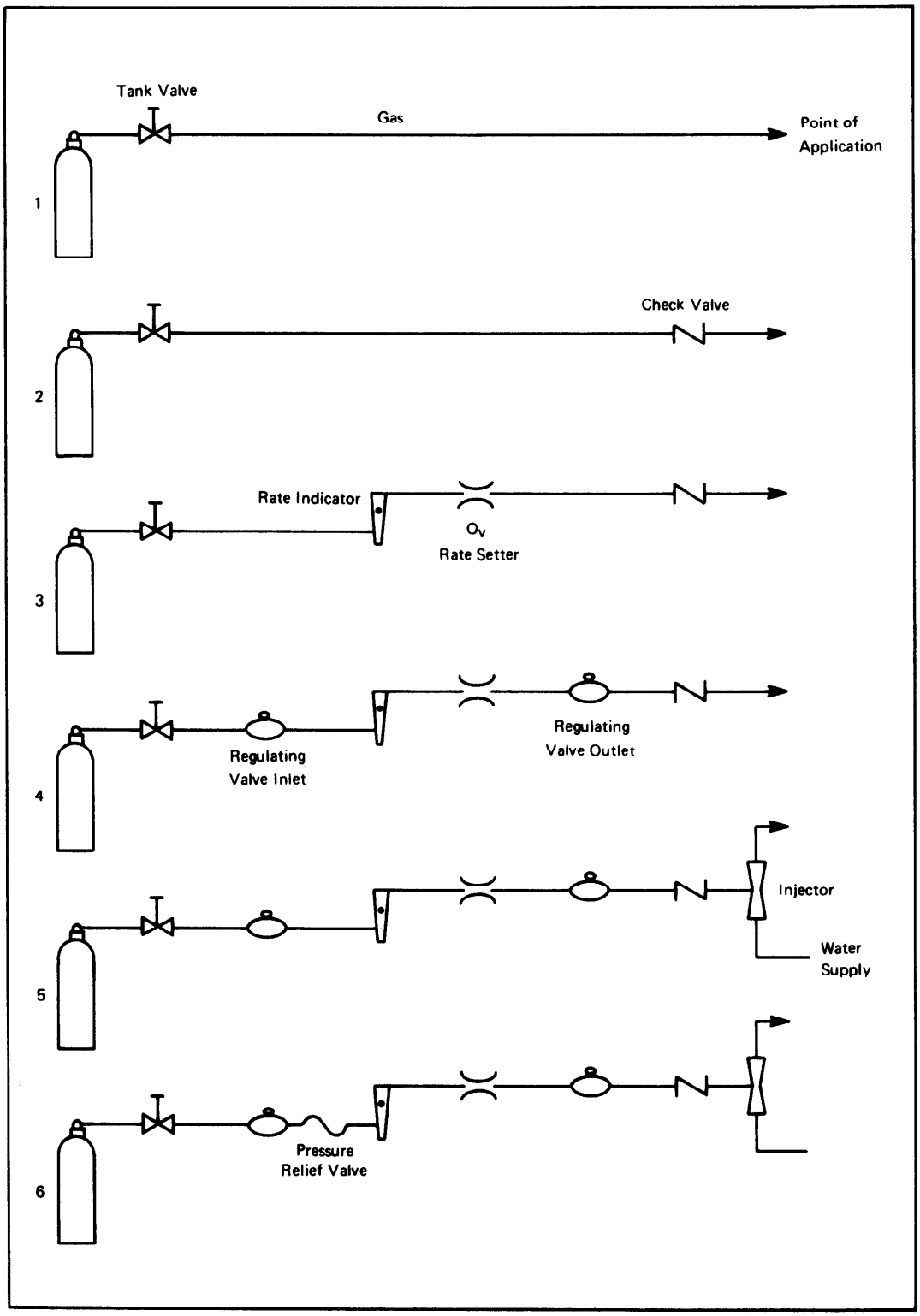


**Fig. 16.** Evolution of Chlorine Feeder

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The result is a full-vacuum-solution-type chlorine feeder that is complete and provides accurate control of chlorine feed; protects personnel, apparatus, and premises; and safely delivers the chlorine to the point of application.

**Methods of control.** There are five basic methods of controlling the operation of chlorine gas feeders.

1. *Manual control.* Refer to Fig. 17. Here the equipment is put into operation and shut down manually by the operator. The feed rate is established by manual adjustment and is continuous at the established rate. The operator must establish his treatment in terms of pounds of chlorine per day from a consideration of the demand and magnitude of the flow to be treated. It follows that in the event of a change in flow or demand, a manual adjustment will have to be made at the chlorine feeder.

This type of control is satisfactory for applications where chlorine demand and flow are reasonably constant or predictable and where an operator is available to make adjustments as necessary.

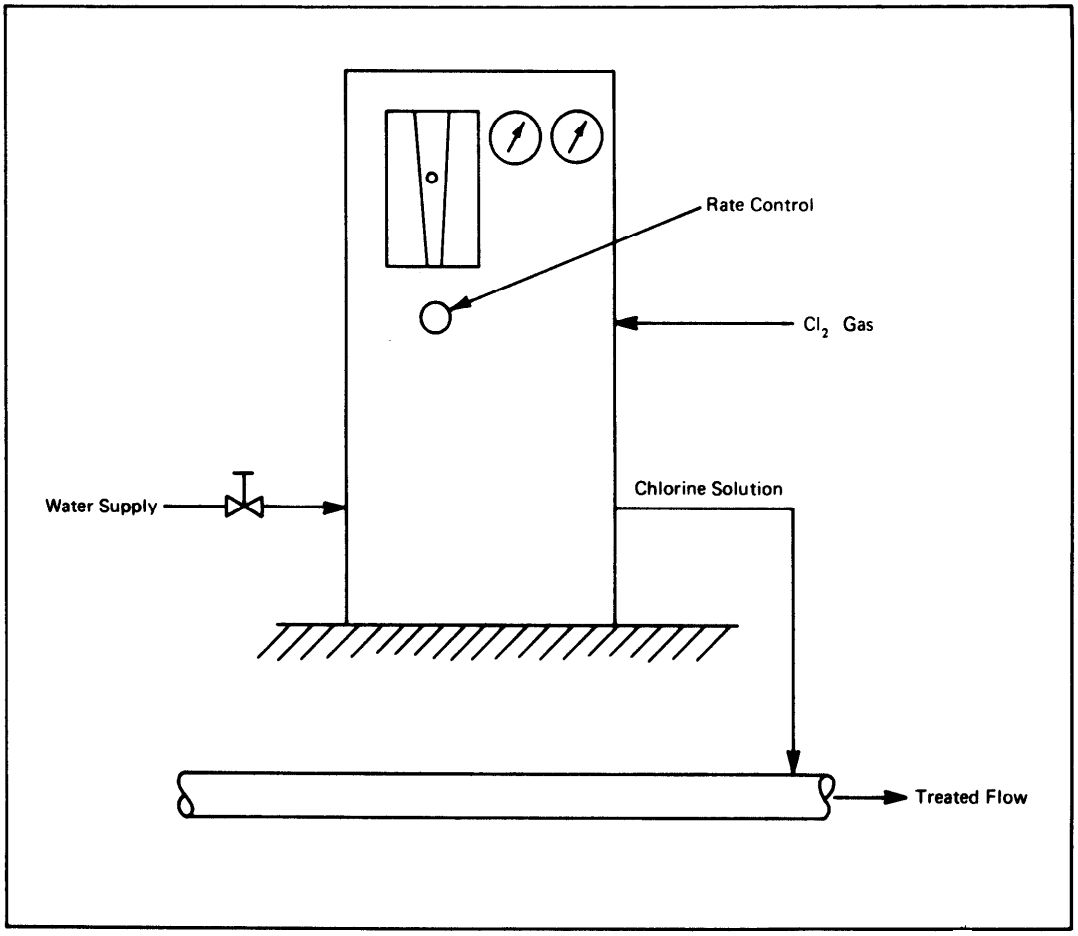


**Fig. 17.** Manual Control of Chlorine Feeder

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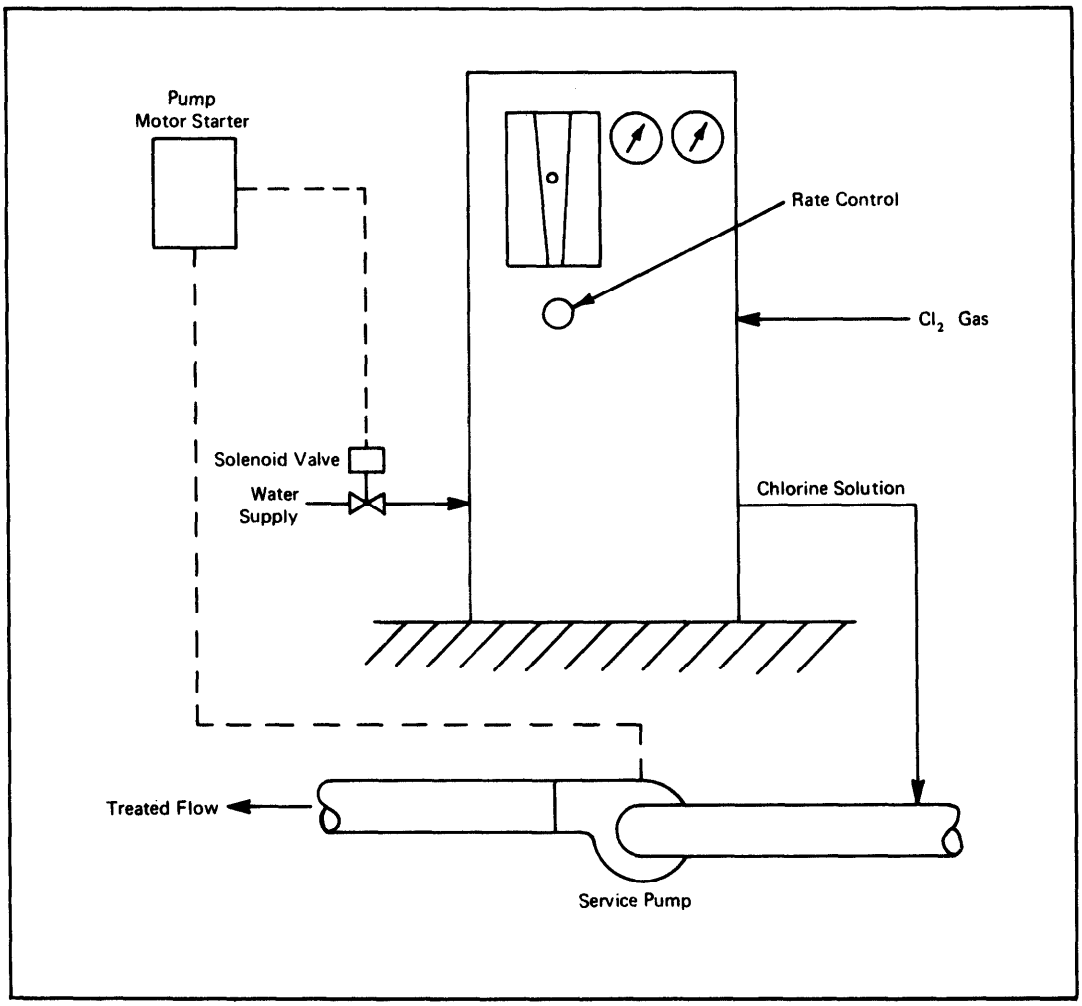


**Fig. 18.** Start-Stop Control of Chlorine Feeder

2. *Start-stop control.* Refer to Fig. 18. This equipment feeds automatically in sequence with pump or timer operation at a manually preset rate. A variation is pump-governed "step control," used in multiple-pump installations in which chlorine feed is automatically synchronized to provide proper dosage for a particular pump or combination of pumps in operation. In start-stop or step control, the operator preselects the rate, and the apparatus then starts and stops in step with the pump or other controlling device to provide chlorine when required at the preselected rate.

3. *Automatic proportional control.* Refer to Fig. 19. The equipment adjusts the chlorine feed rate automatically in accordance with flow changes to provide a constant preestablished dosage for all rates of flow. The feeder operates continuously, the dosage is established by the operator, and the machine then automatically follows the flow, changing the feed rate to maintain a constant dosage. Control is accomplished by metering the main flow and utilizing a pneumatic, vacuum, or electric transmitter to control chlorine feed in exact proportion to the main flow. In each case the pacing signal from the transmitter is proportional to the flow in terms of the characteristic of the chlorine feeder.

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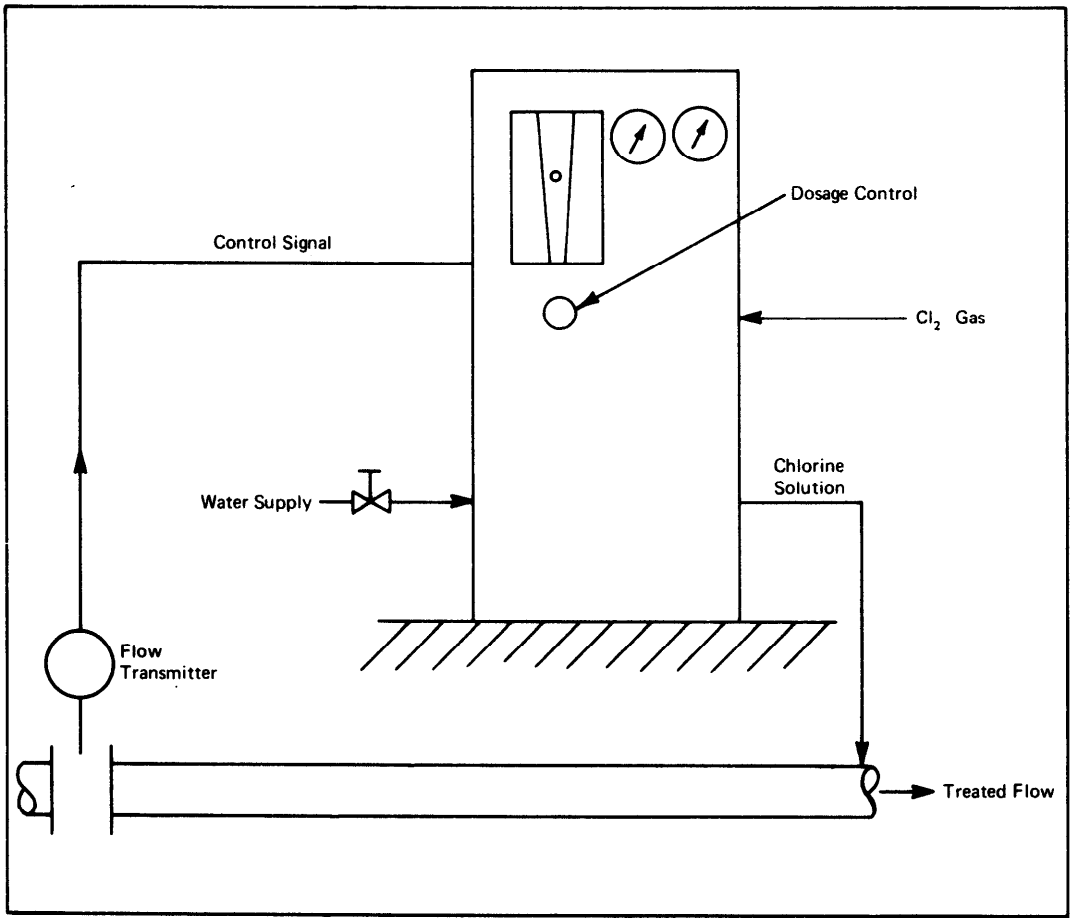


**Fig. 19.** Proportional Pacing of Chlorine Feeder

This type of control is commonly employed in plants where the rate of flow being treated varies.

4. *Automatic residual control.* Refer to Fig. 20. To obtain closed-loop control, the chlorine feeder is used in conjunction with a chlorine residual analyzer-controller. This system, without flow pacing, will maintain a preset residual where flow variations are relatively small and/or slow. The feed rate of the chlorine feeder is controlled by the value of a chlorine residual measured downstream of the point at which the chlorine solution is applied. The analyzer controls the feed rate of the chlorine feeders by actuating a positioner (electric or pneumatic) to control the opening of a variable orifice.

5. *Combined automaticflow and automatic residual control.* Refer to Fig. 21. This system will maintain a preset chlorine residual in water flow independent of chlorine demand and/or flow variations. It includes a flow meter with pacing transmitter, a chlorine residual analyzer-controller, and a chlorine feeder that is responsive to the signals from both the transmitter and analyzers.

This type of control is most effective when flow pacing is used as the primary chlorine feeder control and residual is used to trim the dosage. When the main flow changes, the chlorine feed is changed rapidly by the signal from the pacing transmitter. The fast response from the pacing system bypasses the relatively

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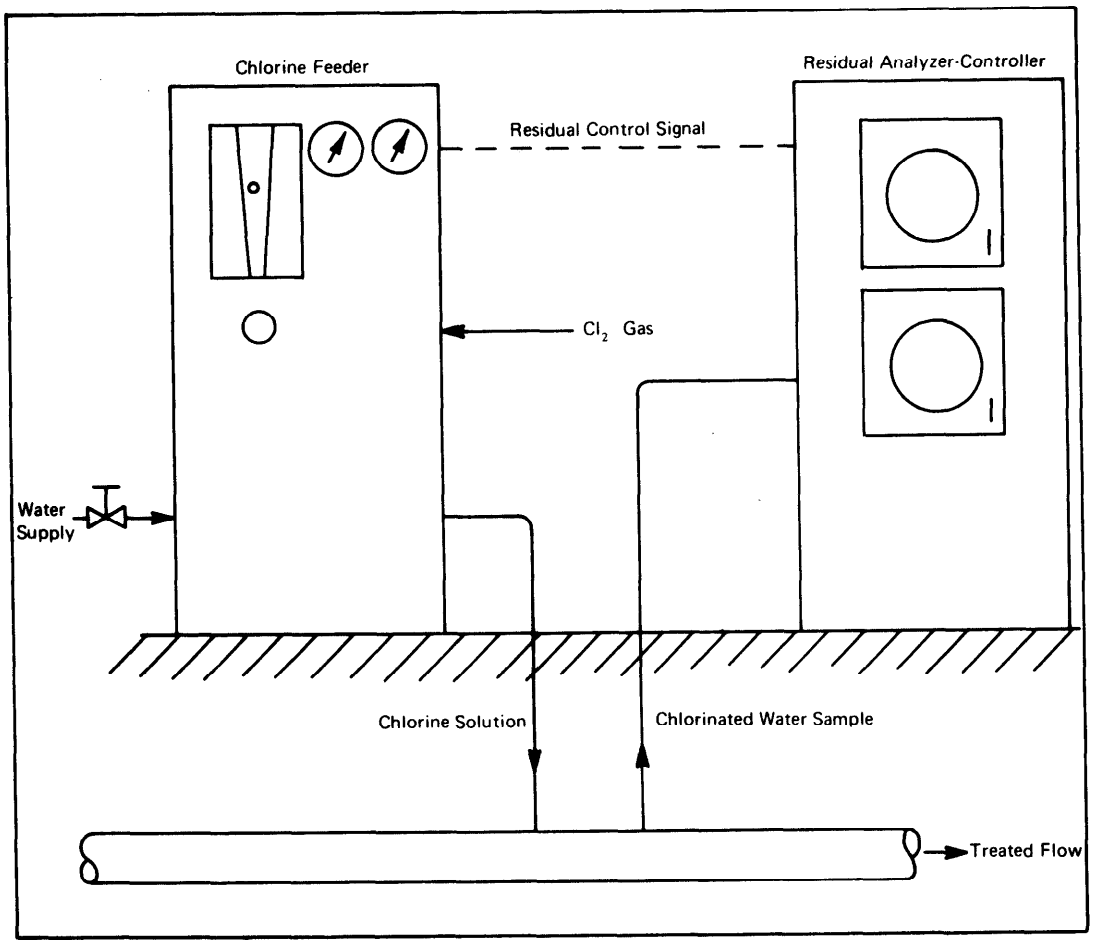


**Fig. 20.** Residual Control of Chlorine Feeder

slow-responding residual control loop; thus the main flow change has little or no effect on the chlorine residual. The residual control loop is exposed to only a small fraction of the potential upset that would be produced if flow pacing were not used.

**Maintenance.** Maintenance begins with a thorough knowledge of the equipment. Every chlorinator is supplied with an instruction book that will include specific steps to follow in servicing. Should the book be lost, manufacturers can supply duplicates. Model and serial number should be included in any request for a replacement.

Since it would be impractical to attempt a discussion of maintenance on all models of every manufacturer, this section covers the four areas most often associated with maintenance requirements and causes of trouble.

1. *Moisture.* Moisture in chlorine is corrosive to ferrous and most nonferrous metals. Most chlorinators use plastic materials in the sections where gas is handled under vacuum. Metal parts or fittings, which are generally exterior to the chlorinator, are header valves, header lines, and flexible connections. When any connection is broken, even for a short time, the openings should be plugged immediately to exclude moisture. Corrosion is internal and not evident upon

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external inspection until failure occurs. A good rule to follow is to exclude moisture from any part of the equipment that is normally exposed to dry chlorine only.

Corrosion products, primarily ferric chloride, are a major cause of chlorinator malfunctioning.

2. *Impurities in chlorine.* Chlorine manufacturers make every attempt to supply a pure product. Even trace amounts of impurities can cause problems if they accumulate. Two compounds are frequently found in chlorinators after continued use. One, ferric chloride, may be present in the chlorine containers or may result when careless operation allows moisture to enter the system. This compound is recognizable as a dark brown, syrupy liquid, and is soluble in water. After chlorinator parts are washed to remove impurity, *they must be dried thoroughly before reassembly.*

The other material, hexachloroethane or a similar compound, is classed as a volatile solid. It tends to deposit in gas lines at points of pressure or temperature drop. This material is not soluble in water, but can be dissolved in trichloroethane, a common industrial solvent.

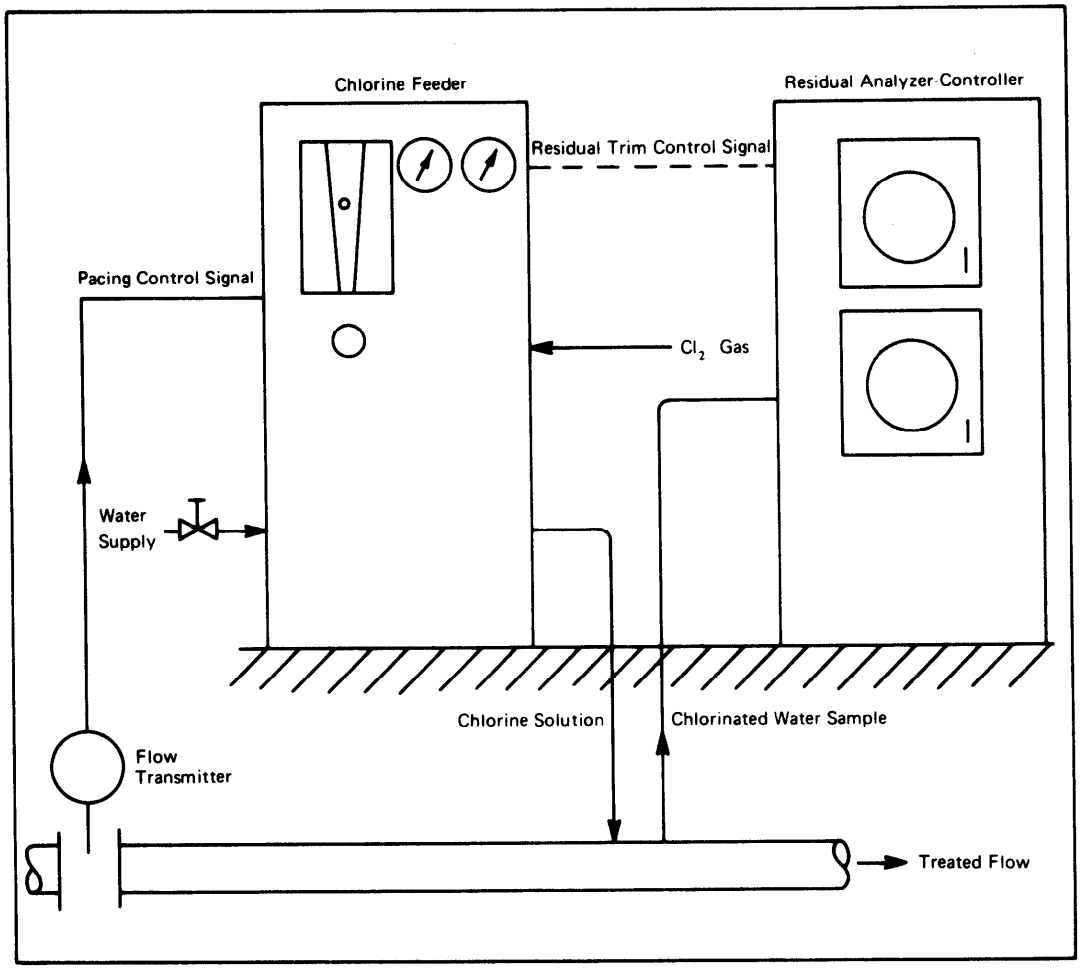


**Fig. 21.** Combined Flow and Residual Control of Chlorine Feeder

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3. *Flexible connections.* Flexible connections (comprising small-diameter metal tubing), used to connect to cylinders or ton containers, deserve special attention. Because they are flexed every time a cylinder is changed, they are subject to metal fatigue. These connections should be changed once a year.

*Each* time a connection is made either to a chlorine container or to the chlorinator, a *new* gasket *must* be used.

4. *Gaskets.* Elastomeric (flexible) materials used for gaskets and O-rings generally become brittle in time. If a gasketed joint is not broken, the gasket may last for years. A regular program of replacement is desirable but guidelines are difficult. Every recommended spare-parts list includes spare gaskets. If swelling or hardening of a gasket is noted, it should be replaced. Distorted or hardened gaskets cannot be properly seated.

## Hypochlorinators

The usual type of equipment used for metering hypochlorite solutions is a positive-displacement diaphragm pump, of which there are three general types.

**Mechanically actuated diaphragm.** This type of pump (Fig. 22) employs a flexible diaphragm to isolate the hypochlorite solution from the operating mechanism. The diaphragm is mechanically attached to an actuator push rod, which is caused to reciprocate by the operating mechanism. The diaphragm is generally circular in shape and convoluted to allow the diaphragm to flex rather than stretch. Usually, a spring return mechanism is employed to return the

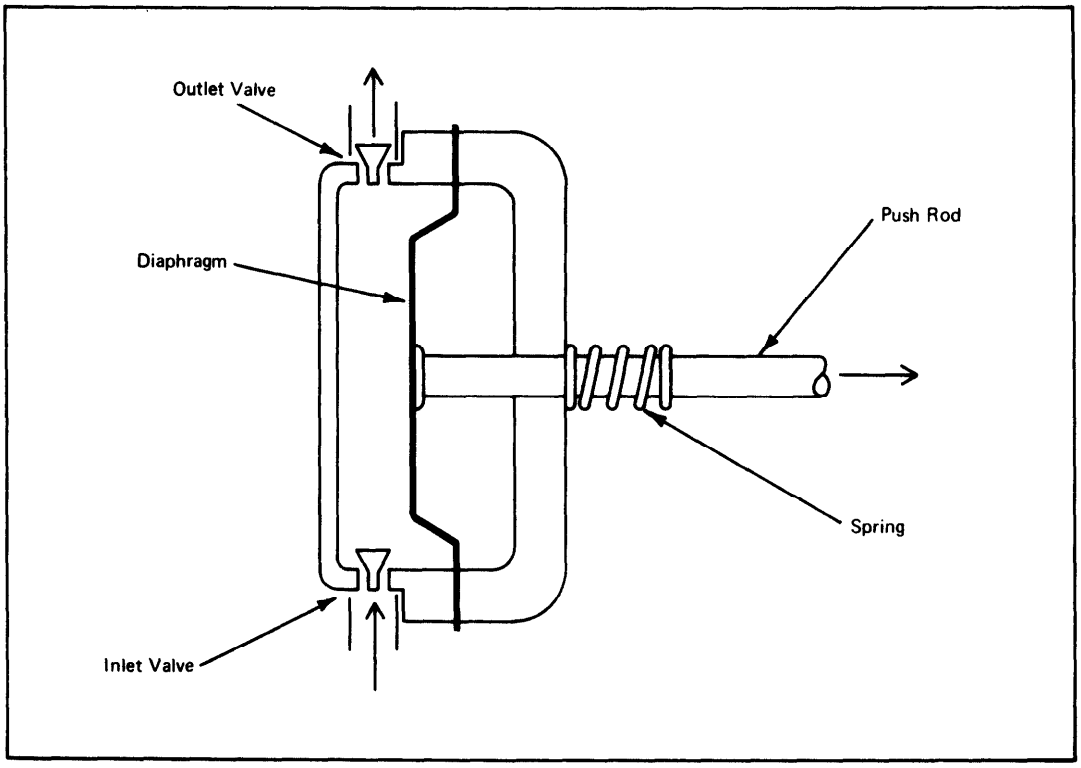


**Fig. 22.** Mechanically Actuated Diaphragm

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diaphragm from its extended position. On the suction stroke, the diaphragm develops a partial vacuum within the liquid head, which closes the discharge check valve, opens the suction check valve, and allows hypochlorite solution to flow in the chamber.

At the end of the suction stroke, the push rod reverses direction and begins the discharge stroke. As the diaphragm is pushed into the feeding chamber, a pressure is created that closes the suction check valve, opens the discharge check valve, and forces the solution out of the head into the discharge line.

At the end of the discharge stroke, the push rod reverses direction and begins another cycle of filling and emptying the feeding head. Since the diaphragm is unsupported on one side, this type of pump is limited to low-discharge-pressure applications (usually pressures not exceeding 125 psig), which, in turn, permit use of different types of basic driving means; i.e., a solenoid may be used to actuate the push rod by push-pull action, or the push rod can be reciprocated by a mechanical device such as an eccentric operating from a gear reducer/electric motor drive, or a secondary diaphragm may be used, operated from a pressurized fluid, either water or air.

**Hydraulically actuated diaphragm (plunger-diaphragm).** The hydraulically actuated diaphragm pump (Fig. 23) employs a diaphragm to isolate the pumped liquid from the operating parts, but, unlike the mechanically actuated diaphragm type, the diaphragm is supported on both sides. Perhaps the simplest generalized explanation of its design is to consider it as a combination of a

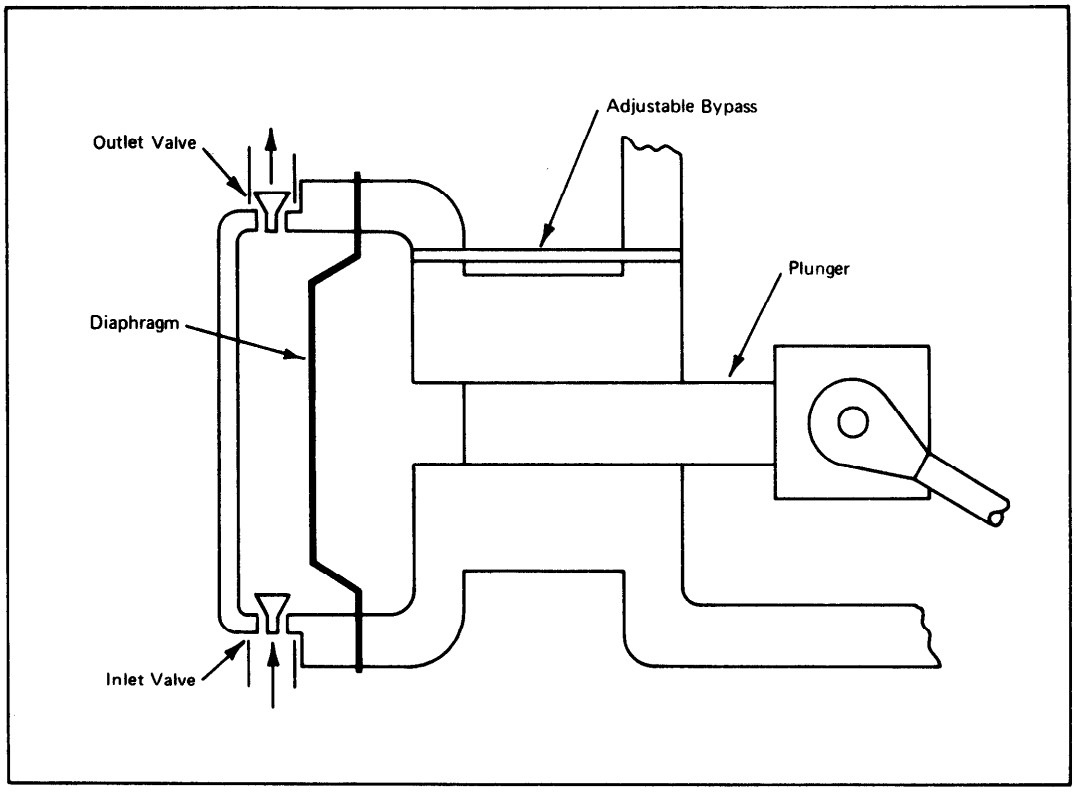


**Fig. 23.** Hydraulically Actuated Diaphragm (Plunger-Diaphragm)

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plunger pump and a diaphragm pump in which the plunger pump is used as a source of pressurized liquid that is applied to the reverse side of the diaphragm, which, in turn, pumps the hypochlorite solution. In this manner, notwithstanding minor flow losses, the diaphragm is said to be balanced, allowing its use at pressures many times higher than is possible with the mechanically actuated diaphragm pump. Also, since the plunger can now be enclosed in a noncorrosive liquid, the stuffing-box-type packing can be replaced with a metal-to-metal seal.

In operation, the plunger is caused to reciprocate in a chamber filled with hydraulic fluid. The displacement of this fluid in turn causes the diaphragm to be flexed, alternately discharging and recharging the pumping chamber. Capacity control is accomplished by varying the volume of hydraulic fluid displaced by the plunger or by bypassing a portion of the fluid displaced by the plunger.

**Hydraulically actuated diaphragm (water operated).** Another type of balanced diaphragm pump is used with a water meter in a pressure line (Fig. 24). A mechanical linkage to the water meter operates a pilot valve that applies line pressure to a power diaphragm. Water pressure is also applied to the back of the pumping diaphragm to equalize the pressure against which the pump must discharge. A spring provides the suction stroke.

In some models, the entire unit is mounted adjacent to the meter with a shaft from the meter head directly connected to the hypochlorinator. In other models, the pilot valve is located on the meter with tubing connected to the pump at a separate mounting.

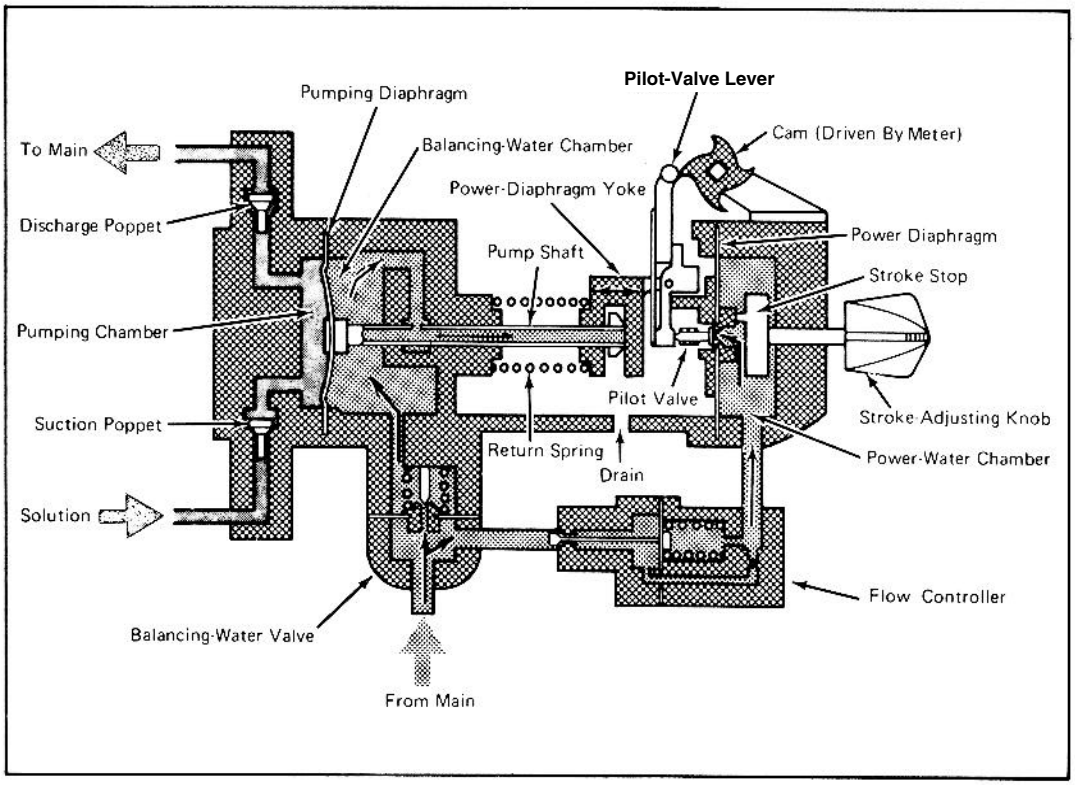


**Fig. 24.** Hydraulically Actuated Diaphragm (Water Operated)

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This type of pump has two distinct advantages: no electric power is required for operation, and the hypochlorite solution is automatically proportioned to the flow of water through the water meter.

**Methods of control.** Basic methods of regulating the feed rate of reciprocating diaphragm pumps involve changing the stroke length or the stroke speed, or both.

1. *Stroke-length adjustment.* It is usually made by restricting the length of the return (suction) stroke. Manually, the procedure requires adjustment of a micrometer screw. Automatic control, proportioning feed rate in response to a water-flow signal, can be accomplished by a pneumatic positioner (usually 3-15 psi air) or an electric or electronic positioner that controls the return-stroke length.

Manual adjustment is usually the only method of dosage control for hypochlorinators with internal electric motors and for water-operated models. Conversely, all hypochlorinators have provisions for manual feed-rate control.

2. *Stroke-speed adjustment.* For hypochlorinators with external motors, stroke-speed adjustment is accomplished with variable-speed drives accepting a pneumatic, electric, or electronic signal proportional to water flow. This adjustment can be made simultaneously with any of the foregoing methods for control of stroke length.

As noted, water-operated hypochlorinators are paced automatically by the water meter, which changes the stroke rate.

The following is a summary of the most commonly used control modes.

| Control Mode | Stroke Length | Stroke Speed |
|---|---|---|
| Manual | Variable for dosage | Constant |
| Start-stop | Variable for dosage | On–off power circuit |
| Automatic proportional to flow (a) | Variable for dosage | Variable-speed drive |
| Automatic proportional to flow (b) | Positioner with ratio control for dosage | Constant |
| Combined flow and residual control | Positioner controlled from analyzer | Variable-speed drive |

**Maintenance.** The parts of hypochlorinators most likely to require maintenance are the diaphragm and the valves. The diaphragm flexes continually in operation. Its life will depend on the strength of the hypochlorite and the pressure it is operating against (in the case of mechanically operated models). The valves and seats may fail as a result of corrosion, of hardening or swelling, or of scale or foreign material preventing proper seating.

All hypochlorite solutions are highly alkaline. This condition frequently causes the deposition of carbonate scale in the pump head, tubing, and the solution diffusor at the point of application. This scale can be readily removed by pumping a dilute (5 per cent) hydrochloric (muriatic) acid solution through the pump, making sure first to flush out the hypochlorite with water.

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## Chlorine Residual Analyzer

The two methods by which a chlorine residual can be continuously analyzed and recorded are (1) amperometric and (2) calorimetric.

This discussion will be limited to the amperometric-type analyzer since it is the one in general use today, It is responsive, reliable, and specific for either free or total residual chlorine. For closed-loop control, it meets the need for specificity and fast responses.

The heart of the amperometric analyzer is a bimetallic cell (refer to Fig. 25) consisting of a gold or platinum measuring electrode and a copper reference electrode. The combination of these two electrodes will produce an electric signal when a small trace of a strong oxidizing agent (such as chlorine) is present. The magnitude of the electric signal will be proportional to the concentration of residual chlorine. This electric signal is fed into a suitable indicator or recorder. In measuring the free residual chlorine; the best results are obtained when the pH of the water sample being measured is between 4 and 4.5. Thus, a pH 4 buffer feeding system is an integral part of the chlorine analyzer. This buffer additive will, in addition, help keep the electrode surfaces clean.

Measurement of total residual chlorine, when combined chlorine is present, requires the addition of potassium iodide to the pH buffer. The potassium iodide

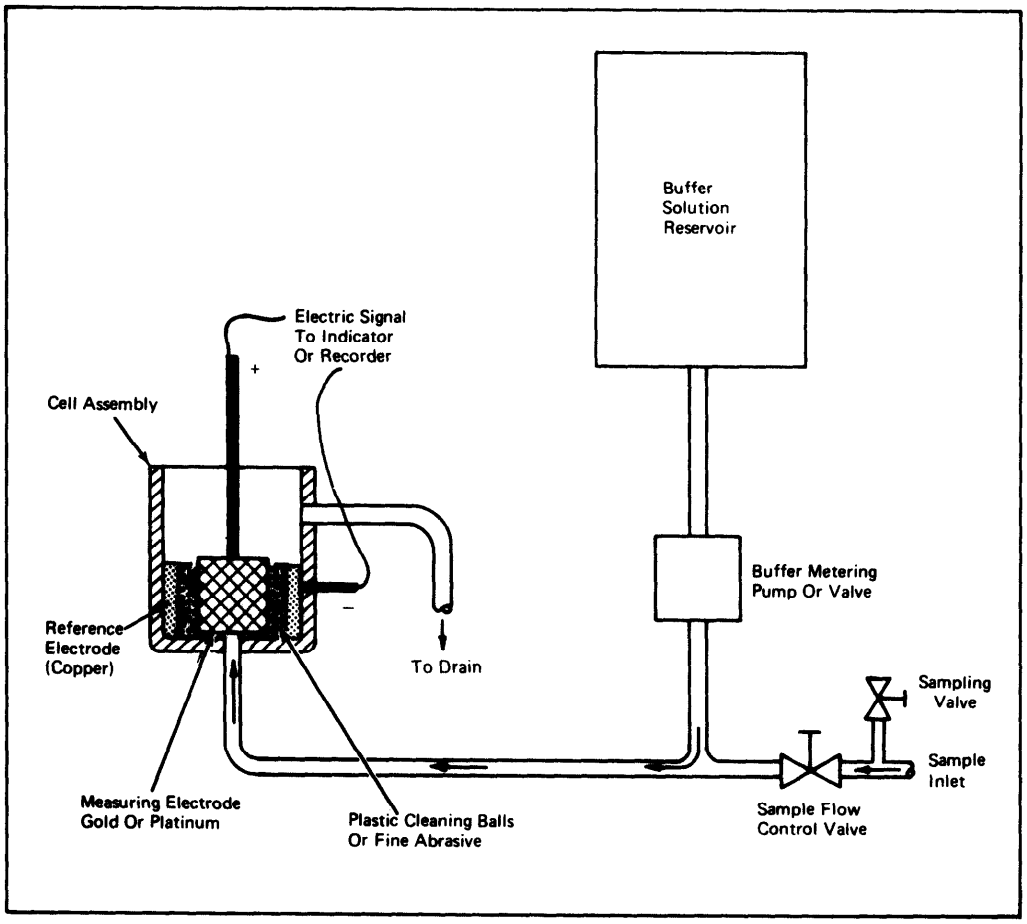


**Fig. 25.** Schematic of Chlorine Residual Analyzer Sensor Assembly

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reacts with the chlorine (both free and combined) to liberate iodine, which will be equal to the total chlorine and will activate the cell in the same manner as chlorine; an electric signal directly proportional to the total residual is then generated.

A number of small, plastic balls or fine abrasive grit is used in the cell to limit contamination of the electrode surfaces. Without such action, the current could slowly decrease.

Analyzers and recorders are available to measure residual chlorine, free or total, in any ranges encountered in water-treatment plants. Suitable controllers may be used to control the dosage rate of a gas chlorinator or hypochlorinator to maintain a preset residual value.

## Chlorine Evaporators

Chlorine vaporizers, better known as evaporators, are needed whenever conditions require the withdrawal of liquid chlorine from the containers. Typically, this action is necessary when the daily chlorine requirements exceed 1,000 to 2,000 lb/24 hr and would require the manifolding of an excessive number of containers or when the source of chlorine is a tank car. Evaporators provide the heat necessary to vaporize or change the liquid chlorine to the gaseous state so that it may be handled in the normal fashion by the other components of the chlorination system.

An evaporator consists of a chlorine pressure vessel to which heat can be applied under controlled conditions. The source of heat may be electricity, steam, or hot water, with several different models and versions available from different manufacturers. The most commonly used models are electrically heated and have maximum rated capacities of 8,000 lb/24 hr. In all cases the source of heat is thermostatically controlled to maintain constant temperature and insure a superheated chlorine gas output.

The need for an evaporator can best be understood by referring to the vapor-pressure curve for chlorine (Fig. 1). The temperature of the chlorine liquid determines the pressure at which the chlorine can be vaporized. This pressure is called the vapor pressure and it increases with increasing temperature. For example, at 30F the vapor pressure is approximately 35 psig; at 70F it is approximately 85 psig. The evaporator serves to provide the heat of vaporization necessary to change the liquid to a gas at that pressure and to superheat the gas to help eliminate liquefaction (chlorine gas condensing to form liquid). It should be clearly understood that the evaporator does *not* cause the chlorine pressure in the system to rise; it simply provides heat for vaporization. In some cases, due to outdoor storage of tank cars in northern climates, it is necessary to air-pad the cars during cold weather to insure adequate liquid chlorine withdrawal rates. In these cases the evaporator also serves to warm the liquid chlorine from the ambient outdoor temperature to the vaporization temperature. For example, the liquid chlorine in a tank car located outside in a northern area may be at 0F and the vapor pressure would be about 12 psig, which may be insufficient for proper operation of the chlorination equipment. If so, the tank car can be pressured with clean, very dry air. If the air-pad pressure

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is 50 psig, the OF liquid chlorine must be heated in the evaporator to the corresponding vaporization temperature of 43F.

A schematic of a typical electrically heated evaporator is shown in Fig. 26 and will be used to explain the operation of this type of unit. The evaporator consists of a metal pressure vessel immersed in a water bath heated by a thermostatically controlled electric immersion heater. Proprietary designs differ, but the basic operating principle is the same. The heaters may be located at the bottom of the water jacket beneath the cylinder or suspended alongside it. Chlorine liquid is introduced into the pressure vessel via the long entry tube that reaches nearly to the bottom of the vessel. Alternately, it may be introduced through the bottom.

The liquid absorbs heat from the water chamber through the wall of the pressure vessel until it reaches the vaporization temperature and boils, releasing chlorine gas. With the chlorination system in operation, gas is withdrawn from the pressure vessel, allowing more liquid to enter the system

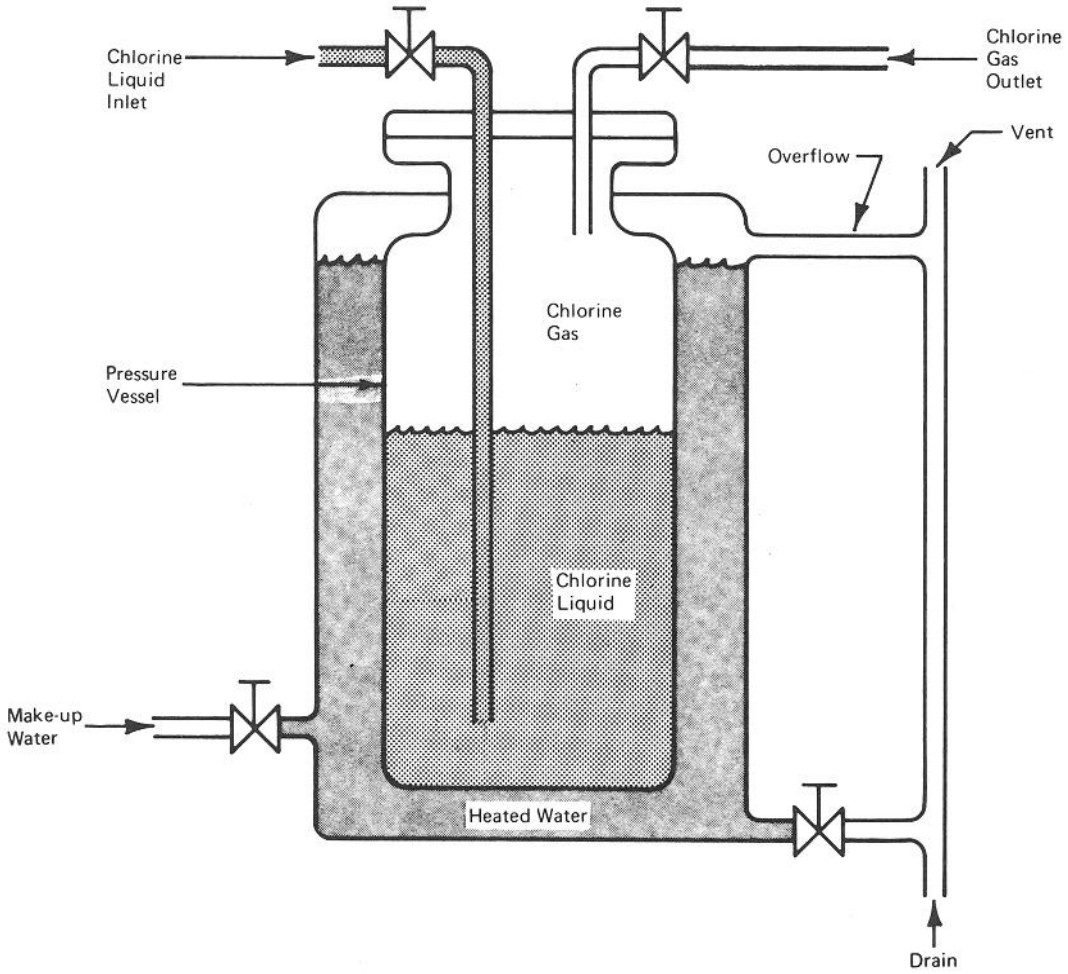


**Fig. 24.** Schematic of Chlorine Evaporator

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and continue the process. The gas leaving the vessel contacts the hot upper wall and causes the gas to be heated to a higher or superheated temperature. Baffles may be used to assist heat transfer.

As heat is absorbed by the chlorine, the water is cooled until it drops below the control thermostat setting and causes the electric immersion heater to be actuated. The heater remains on until the water reaches the upper limit of the thermostat. At this point, the heater is shut off, with this cycling continuing as long as the evaporator is in use. As chlorine is vaporized, any impurities contained in the liquid are left behind, coating the inside surface of the pressure vessel. This coating acts as an insulator that inhibits heat transfer from the water to the chlorine and necessitates higher and higher water-bath temperature settings. Eventually, the pressure vessel will require cleaning as the evaporator will be unable to vaporize the desired amount of chlorine gas at the desired degree of superheat. The frequency of cleaning will vary from installation to installation and is a function of chlorine purity and the chlorine feed rate. The evaporator manufacturer's cleaning instructions should be followed closely for best results.

Typically, evaporators are supplied with various gages and controls, in addition to the control thermostat, to permit simple, safe operation. Taken individually, they include the following:

*Control* thermostat senses water temperature and turns heater on and off to control water-bath temperature.

*Water-level gage* is a sight glass that permits the operator to observe the water-bath level.

*Chlorine-gas-pressure gage* indicates chlorine gas pressure within the pressure vessel.

*Chlorine-gas-temperature gage* indicates temperature of the superheated chlorine gas leaving the pressure vessel.

*Water-temperature gage* indicates the temperature of the water bath.

*Low-water-temperature switch* actuates at a preset low-water bath temperature, which normally indicates loss of the heater system. The switch may be used to actuate an alarm and automatically shut the chlorine gas shutoff valve to avoid the pulling of liquid chlorine into the gas system.

*High-water-temperature switch* actuates at a preset high-water bath temperature. Excessive temperature may be caused by failure of the heater controls, which allows the heater to remain on, or by loss of water from the water bath. The switch may be used to actuate an alarm and turn off power to the electric heater.

*Low-water-level switch* actuates at a preset low-water bath level, caused by a failure of the water-supply system. Switches may be used to actuate an alarm and automatically open a water valve.

*Magnetic heater contactor* acts in response to the control thermostat to energize or deenergize the electric immersion heater.

*Cathodic protection* protects from corrosion all metal surfaces of the evaporator in contact with water.

*Vent alarm switch* actuates whenever gas flow occurs in the vent line and indicates that the gas-pressure-relief valve has opened.

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*Liquid chlorine expansion tank* protects the liquid chlorine piping system from damage due to overpressure by offering a chamber into which the liquid can expand. This tank is strongly recommended for installation in any section of liquid piping which may be purposely or accidentally isolated by closing valves. (See Fig. 27)

*Water purgemeter* provides positive indication of the water flow to the evaporator water bath and permits adjustment of flow rate.

Some or all of the aforementioned controls, gages, and accessories may be supplied with an evaporator.

## Chlorine Pressure-Reducing Valves

Chlorine pressure-reducing valves are used to eliminate the problem of liquefaction (chlorine gas condensing to form liquid chlorine). Referring to the vapor pressure curve for chlorine (Fig. 1), one can see that lowering the gas pressure acts to lower the temperature at which liquefaction can occur. For example, a pressure-reducing valve installed and set at 20 psig will prevent liquefaction from occurring downstream of the valve at temperatures above 10F. A lower setting would further reduce the liquefaction temperature. The elimination of the possibility of chlorine liquid reaching the chlorinators is a

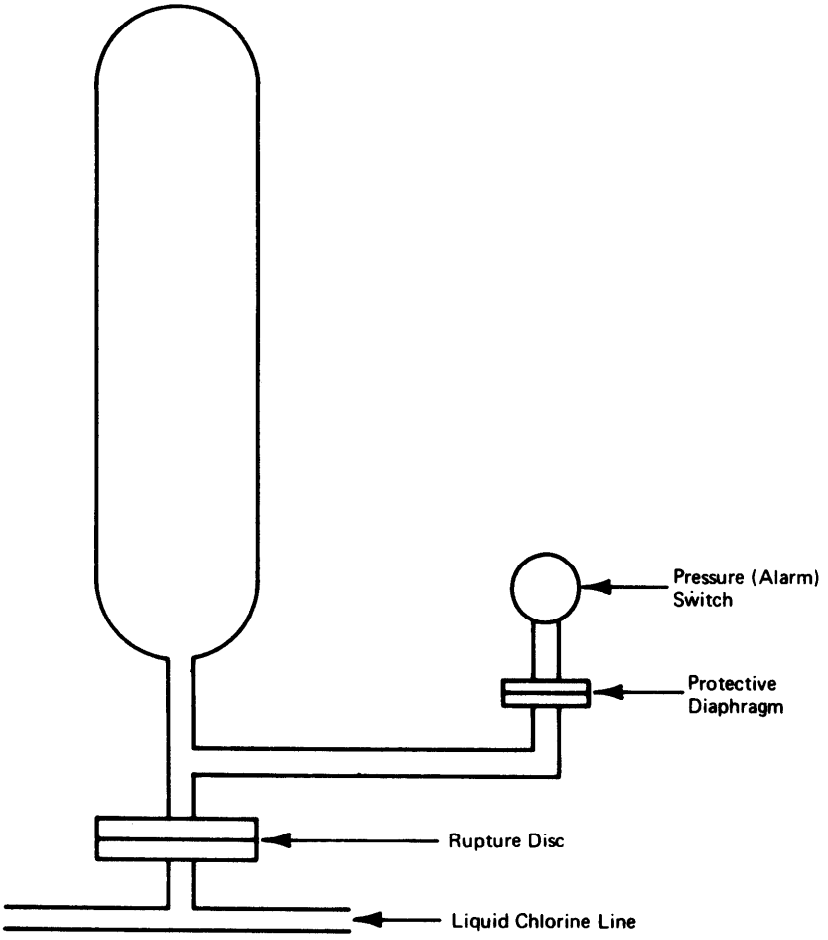


**Fig. 27.** Chlorine Expansion Chamber

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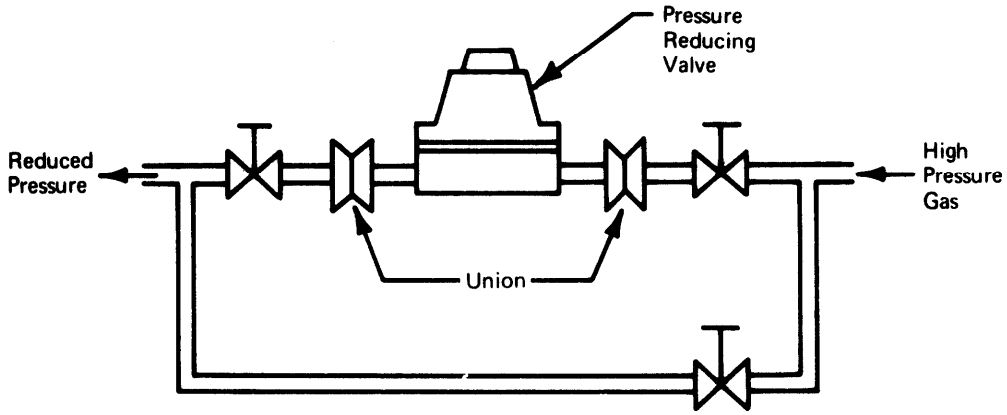


**Fig. 28.** Pressure-Reducing Valve with Bypass

major step in preventing equipment malfunction and makes the pressure-reducing valve a most valuable part of a chlorination system.

Chlorine pressure-reducing valves are available in both self-actuated and automatic versions. The self-actuated style is spring-actuated and may be set to maintain a relatively constant lower downstream pressure. An electrically operated version combines the features of the self-actuated unit with an on-off electric actuator, permitting the valve to be opened and closed electrically. Hydraulically and pneumatically actuated types are also available. They permit the downstream pressure to be controlled and also provide automatic closing of the valve by removing the hydraulic or pneumatic actuating pressure.

All types of pressure-reducing valves are available in materials suitable for use on chlorine and should be installed as far upstream in the gas piping system as possible to provide the maximum protection. It is also good practice to install a liquid chlorine trap or drip leg in the piping immediately upstream of the pressure-reducing valve to prevent liquid chlorine from reaching the valve and flashing across the valve seat. This condition results in poor pressure regulation and valve flutter and should be avoided. Figure 28 shows a typical installation of a pressure-reducing valve with bypass valves to facilitate servicing.

## Chlorine Weighing Scales

Chlorine weighing scales form an integral part of a chlorination system since they permit the operator to determine easily the total amount of chlorine dispensed during any period by noting the change in weight of the cylinders in use. Scales fall into two basic categories: portable and dormant, with several variations of each in use. The term portable as applied to chlorine scales indicates the presence of wheels that permit the moving of the scale from place to place. Due to their nature, most scales of this type are limited in weighing capacity. Dormant scales, on the other hand, are designed to be fixed in place. Both portable and dormant-type scales are available in double-beam or direct-reading dial styles. In either case, the tare weight (weight of the empty cylinder) can be set in so that the weight indicated is the actual chlorine weight. The more expensive dial types indicate the weight of chlorine directly on the dial, whereas the beam scales require rebalancing for each reading. These scales may be mounted on the floor or recessed in pits to facilitate loading and unloading of

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containers. Styles are available for weighing of upright 100-lb and 150-lb cylinders. Ton containers are positioned horizontally on cradles.

Load-cell-type scales are meeting with increasing favor among chlorine users, particularly those using ton containers. These scales require less space than dial or beam scales and have fewer moving parts. They typically consist of structural members designed to support or cradle the container(s). The weight of the containers, chlorine, and structure is sensed by the load cell and transmitted to a wall-mounted dial-type gage with adjustments permitting the zeroing out of the cylinder and structural weights.

Scales are frequently supplied with loss-of-weight recorders, which, as the name implies, record the decrease in the amount of chlorine on the scale. Switches may be supplied in the recorder. They are set to actuate at a minimum chlorine weight and alert operating personnel to the need for changing cylinders.

### Chlorine Flow-Recording Systems

Numerous types of chlorine flow-recording systems make use of the wide variety of flowmeters that are available. This text will not describe any of these systems in detail, but will briefly review their uses and limitations. First, it may clearly be stated that chlorine flow-recording systems are not to be considered as direct substitutes for weighing scales. Because of the wide variations in chlorine flow rate on a particular installation and the changes in gas density due to changes in temperature and pressure, the system accuracy cannot approach that of a scale. Some of the errors can be compensated for; however, this makes the systems more complex and more difficult to calibrate and operate. In many cases, systems of this type are used successfully in spite of their limitations because they permit the recording of the split in chlorine gas flows, from a common source to several points of application, without the need for multiple scales. Also, it is frequently felt that when the source of chlorine is to be a railroad tank car, these system limitations are small compared with the high cost and winter maintenance problems associated with a railroad car scale located out of doors.

### Automatic Changeover Systems

Increased emphasis on the need for uninterrupted chlorination has led to the use of automatic changeover systems, particularly at unmanned stations. The basic concept of these systems is to switch from a depleted source of chlorine to a standby source automatically without the presence of an operator. Several methods have been used to accomplish this. One system consists of electrically operated chlorine shutoff valves, actuated by a chlorine-pressure switch that senses the loss of chlorine pressure due to empty cylinders. Another system uses two pressure-reducing valves, each attached to its own source of chlorine and manifolded on the downstream sides. The pressure settings of the two valves are adjusted so that the valves control at pressures approximately 5 psig apart. Since a pressure-reducing valve will not open until the downstream pressure is lower than its setting, the valve with the higher setting opens first, allowing gas to flow through the valve from its source. This process continues until the first source is depleted and the downstream pressure drops to the setting of the second valve, at which point it opens and chlorine flows from the standby source.

The recent development of small cylinder-mounted chlorinators has added

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more types of automatic changeover systems to the marketplace. It is not necessary to detail the operation of each, but merely to state that they meet the basic need of permitting continuous chlorine feed in a simple, inexpensive manner for even the smallest gas chlorination facilities.

## Chlorine Detectors

Continuous monitoring of atmospheres in areas where chlorine is stored and fed is an important aspect of any safety program. Instruments for this purpose are called chlorine detectors, which are not to be confused with the detectors used for measuring residual chlorine in water.

Concentrations are expressed as ppm by volume in air, not as parts per million parts by weight, as the expression is used to denote concentrations in water. For comparison, 1 ppm of chlorine by volume in air is equivalent to 3 mg/cu m of air. The threshold of odor perception is about 3 ppm.

Two types of detectors are available (Fig. 29). In type 1, the air to be sampled is directed to a rotating drum covered with a strip of sensitized paper. The paper is white, and light is reflected from it to a photoelectric cell. The current from the cell is amplified and used to keep an electric relay open in an alarm circuit. If the air sample contains chlorine, the paper darkens, the light is absorbed, the current from the photoelectric cell drops below that required to keep the relay open, and thus the alarm circuit is energized.

In the second type (Fig. 29), air from the point or points of sampling is drawn to the detector by an air pump, through a filter and flowmeter that indicates the sample flow rate. The air sample is directed to an electrochemical sensing cell, the electric output of which increases with the presence of chlorine. A meter movement is incorporated to indicate visually the strength of the chlorine in air content, and an adjustable switch is included to provide a contact closure for remote audible/visual alarms or an exhaust fan.

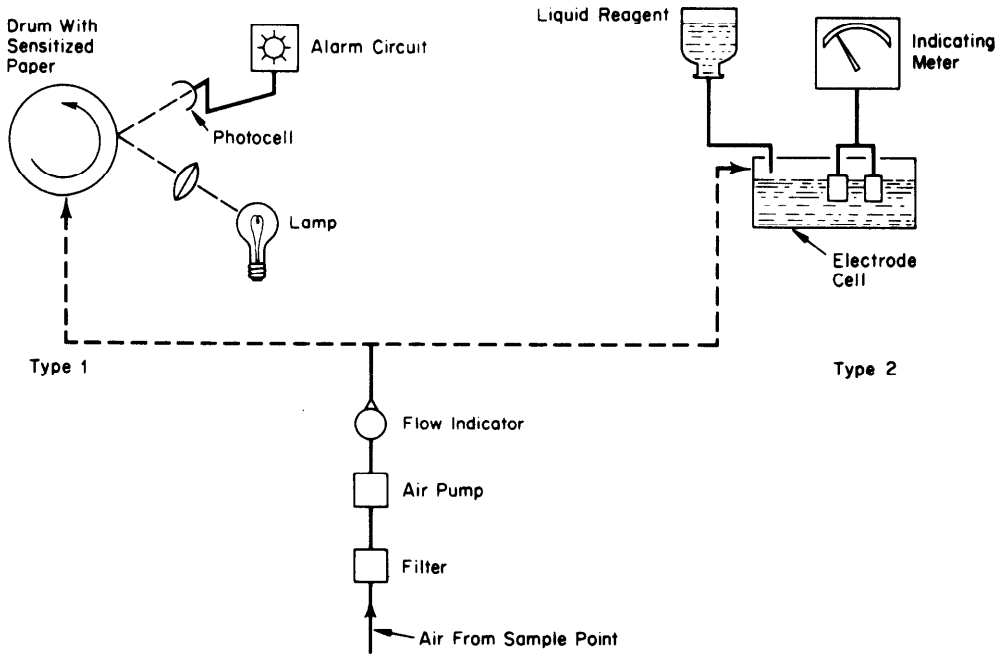


**Fig. 29.** Schematic of Chlorine Detector

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APPENDIX 1

# Calculations

The *dosage* of chlorine is the amount applied to a specific quantity of water. Dosage is usually expressed as milligrams per liter (mg/l) or its equivalent, parts per million (ppm). (For the purpose of the following calculations, milligrams per liter will be considered a dimensionless quantity.)

The *dosage rate* is usually expressed as pounds per 24 hr and is determined by multiplying the dosage in mg/l times 8.33, the weight in pounds of one gallon of water, times the water flow rate in millions of gallons per day (dosage × 8.33 × mgd).

Assume a residual of 0.2mg/l is to be maintained after a contact time of 10 min in water having a 10 min demand of 0.4 mg/l.

1. What dosage of liquid chlorine will be required to treat 1 mil gal?
   *Solution:*

$$\text{demand } (0.4) + \text{residual } (0.2) = 0.6 \text{ mg/l}$$

2. How many pounds will be required?
   *Solution:*

$$0.6 \text{ mg/l} \times 8.33 \times 1 \text{ mil gal} = 5 \text{ lb}$$

3. What dosage rate will be required to treat 6 mgd?
   *Solution:*

$$5 \text{ lb} \times 6 \text{ mgd} = 30 \text{ lb/24 hr}$$

   or

$$0.6 \text{ mg/l} \times 8.33 \times 6 \text{ mgd} = 30 \text{ lb/24 hr}$$

4. How many gallons of 15 per cent sodium hypochlorite will be required?
   *Solution:* One gal of 15 per cent sodium hypochlorite contains 1.25 lb available chlorine according to the table:



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**Available Chlorine in Sodium Hypochlorite***

| Per Cent Available Chlorine | 1 | 5 | 10 | 15 |
|---|---|---|---|---|
| Pounds of Available Chlorine per Gallon | 0.083 | 0.42 | 0.83 | 1.25 |

*Based on weight per unit volume.

$$\frac{\text{lb } Cl_2/\text{mil gal required}}{\text{lb/gal available } Cl_2} \times \text{mgd} = \text{gal/24 hr}$$

$$\frac{5}{1.25} \times 6 \text{ mgd} = 24 \text{ gal/24 hr}$$

5. How many pounds of 70 per cent hypochlorite powder will be required?

*Solution:*

$$\frac{\text{lb } Cl_2/\text{ mil gal required}}{\frac{\%\text{ available } Cl_2}{100}} \times \text{mgd} = \text{lb dry powder/24 hr}$$

$$\frac{5}{\frac{70}{100}} \times 6 \text{ mgd} = 43 \text{ lb/24 hr}$$

## Preparation of Solutions

1. How many gallons of a 15 per cent solution must be diluted with water to prepare 55 gal of a 2 per cent solution?

*Solution:*

$$\frac{\text{gal dil. sol.} \times \%\text{ dil. sol.}}{\%\text{ conc. sol.}} = \text{gal conc. sol.}$$

$$\frac{55 \text{ gal} \times 2\%}{15\%} = 7.33 \text{ gal conc. sol.}$$

2. How many pounds of dry hypochlorite powder containing 70 per cent available chlorine must be dissolved in 55 gal of water to make a 2 per cent solution?

*Solution:*

$$\frac{\text{lb of water} \times \frac{\%\text{ sol.}}{100}}{\frac{\%\text{ strength}}{100}} = \text{lb hypochlorite powder}$$

$$\frac{55 \text{ gal} \times 8.33 \text{ lb} \times \frac{2}{100}}{\frac{70}{100}} = 13 \text{ lb hypochlorite powder}$$

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## Determination of Dosage Applied

1. Determine the dosage if 6 mgd of water is chlorinated with liquid chlorine at a rate of 120 lb/24 hr.

*Solution.*

$$\frac{\frac{\text{lb/24 hr}}{8.33}}{\text{mgd}} = \text{mg/l}$$

$$\frac{\frac{120}{8.33}}{6} = 2.4 \text{ mg/l}$$

2. Determine the dosage if 6 mgd of water is chlorinated with a *2* per cent solution of hypochlorite at a rate of 180 gal/24 hr.

*Solution.*

$$\text{gal sol.} \times 8.33 \times \frac{\% \text{ sol.}}{100} = \text{lb chlorine}$$

$$180 \times 8.33 \times \frac{2}{100} = 30 \text{ lb chlorine}$$

from solution to 1. (above)

$$\frac{\frac{30}{8.33}}{6} = 0.6 \text{ mg/l}$$

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APPENDIX 2

# Glossary of Terms

**Absorption:** The taking up of one substance into the body of another.

**Acid:** (1) A substance that tends to lose a proton. (2) A substance that dissolves in water with the formation of hydrogen ions. (3) A substance containing hydrogen that may be replaced by metals to form salts.

**Alkali:** Any of certain soluble salts, principally of sodium, potassium, magnesium, and calcium, that have the property of combining with acids to form neutral salts and may be used in chemical processes such as water or wastewater treatment.

**Alkalinity:** The capacity of water to neutralize acids, a property imparted by the water's content of carbonates, bicarbonates, hydroxides, and occasionally borates, silicates, and phosphates. Alkalinity is expressed in milligrams per liter of equivalent calcium carbonate.

**Ammonia:** A colorless, pungent, suffocating gas, $NH_3$, very soluble in water. Used with chlorine in the "combined residual chlorination" method of disinfection.

**Ammoniator:** Apparatus used for applying ammonia or ammonium compounds to water.

**Antichlors:** Reagents such as sulfur dioxide and bisulfite that can remove excess chlorine.

**Automatic Controller:** The mechanism that measures the value of the variable and corrects or limits the deviation from the desired value.

**Available Chlorine:** A term used in rating chlorinated lime and hypochlorites as to their total oxidizing power.

**Base:** A compound that dissociates in aqueous solution to yield hydroxyl ions.

**Bleach:** A compound containing chlorine, usually combined with calcium or sodium, in a dry or liquid form. See chlorinated lime.

**Bleach, Powder:** See chlorinated lime.



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**BOD:** (1) Abbreviation for biochemical oxygen demand. The quantity of oxygen used in the biochemical oxidation of organic matter in a specified time, at a specified temperature, and under specified conditions. (2) A standard test used in assessing wastewater strength.

**Breakpoint Chlorination:** Addition of chlorine to water or wastewater until the chlorine demand has been satisfied and further additions result in a residual that is directly proportional to the amount added beyond the breakpoint.

**Buffer:** Any of certain combinations of chemicals used to stabilize the pH values of alkalinities of solutions.

**Calibration:** A procedure by which a recorder or indicator is matched to a known value of a variable.

**Chemical Feeder:** A device for dispensing a chemical at a predetermined rate for the treatment of water or wastewater. Change in rate of feed may be effected manually or automatically by flow-rate changes. Feeders are designed for solids, liquids, or gases.

**Chemical Treatment:** Any process involving the addition of chemicals to obtain a desired result.

**Chloramine:** Compounds of organic amines or inorganic ammonia and chlorine.

**Chloride:** A compound containing the (C1− ) radical.

**Chlorinated Lime:** A combination of slaked lime and chlorine gas in which calcium oxychloride is the essential ingredient. When dissolved in water, the calcium oxychloride decomposes to provide calcium hypochlorite and calcium chloride.

**Chlorination:** The application of chlorine to water, or wastewater, generally for the purpose of disinfection, but frequently for accomplishing other biological or chemical results.

**Chlorinator:** A device to apply chlorine to water at a known, controlled rate.

**Chlorine:** An element ordinarily existing as a greenish yellow gas about 2.5 times as heavy as air. Under atmospheric pressure and at a temperature of − 30.0F, the gas becomes an amber liquid about 1.5 times as heavy as water. The chemical symbol of chlorine is C1, its atomic weight is 35.457, and its molecular weight is 70.914.

**Chlorine-Ammonia Process:** The application of chlorine and ammonia to water, or of ammonia to water containing chlorine, to provide combined residual chlorination.

**Chlorine Demand:** The difference between the amount of chlorine added to water or wastewater and the amount of residual chlorine remaining at the end of a specified contact period. The demand for any given water ‘varies with the amount of chlorine applied, time of contact, and temperature.

**Chlorine Dioxide:** A heavy, reddish-yellow gas that has a pungent disagreeable odor and is soluble in water. The compound has disinfecting properties, but is used primarily for taste and odor control; $C1O_2$.

**Chlorine Gas Feeders:**

*Direct-Feed Type:* Chlorine gas under pressure is delivered directly to the point of application.

*Solution Type:* Chlorine gas is dissolved with a minimum amount of water in an injector or eductor and delivered in solution to the point of application.

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*Vacuum Type:* Solution-type feeder in which chlorine gas is maintained under vacuum throughout the apparatus. An ejector is employed into which the gas is drawn to provide solution feed and create vacuum. Loss of vacuum in the system causes automatic shutoff.

**Chlorine Hydrate:** A yellowish compound formed when chlorine gas comes in contact with water at 49 F or lower. The compound is also called chlorine ice.

**Chloriae Residual:** The total amount of chlorine (combined and free available chlorine) remaining in water at the end of a specified contact period following chlorination.

**Clarifier:** A structure in which the solids in water are permitted to settle.

**Closed-Loop Control System:** A control system in which the control action is dependent on the output.

**Combined Available Residual Chlorine:** That portion of the total residual chlorine remaining in water at the end of a specified contact period that will react chemically and biologically as chloramines or organic compounds.

**Combined Residual Chlorination:** The application of chlorine to water or wastewater to produce directly, or with the addition of ammonia, combined available residual chlorine.

**Compound Loop Control:** A chlorination process that maintains a set chlorine residual despite changes in flow or chlorine requirement.

**Concentration:** A measure of the amount of dissolved substances contained per unit volume of solution. Concentration may be expressed as grains per gallon, pounds per million gallons, or milligrams per liter.

**Control System:** An arrangement of physical components connected or related in such a manner as to command, direct, or regulate itself or another system.

**Corrosion:** The deterioration or destruction of a substance or material by chemical action, frequently induced by electrochemical processes, the action proceeding inward from the surface.

**Dead Zone:** The range through which the variable can change without the change being sensed by the instrument.

**Dechlorination:** The partial or complete reduction of residual chlorine in a liquid by any chemical or physical process.

**Decomposition:** Breakdown of organic matter to a more stable form.

**Disinfection:** The art of killing the larger portion of microorganisms in or on a substance, with the probability that all pathogenic bacteria are killed by the agent used.

**Ejector:** A device for moving a fluid or solid by entraining it in a high-velocity stream of air or water.

**Feedback:** The property of a closed-loop system that permits the output to be compared with the input of the system so that the appropriate control action may be formed as some function of the output and input.

**Feeder, Chemical, Dry:** A mechanical device for applying dry chemicals to water at a rate controlled manually or automatically.

**Feeder, Chemical, Solution:** A mechanical device for applying chemicals in liquid form to water at a rate controlled manually or automatically.

**Floc:** Small gelatinous masses formed in a liquid by the reaction of a coagulant added thereto, through biochemical processes, or by agglomeration.

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**Free Available Residual Chlorine:** That portion of the total residual chlorine remaining in the chlorinated water or waste at the end of a specified contact period that will react chemically and biologically as hypochlorous acid or hypochlorite ion.

**Free Residual Chlorination:** The application of chlorine to water or wastewater to produce directly, or through the destruction of ammonia or of certain organic nitrogenous compounds, a free available chlorine residual. (See breakpoint chlorination.)

**gpd:** Gallons per day.

**Gram:** A metric unit of mass defined as one thousandth of a kilogram; the gram is practically equal to the weight of a cubic centimeter of water.

**Hydrochloric Acid:** When absorbed in water, hydrogen chloride gas forms hydrochloric acid, HC1. It is also called muriatic acid.

**Hydrogen Ion Concentration:** See pH.

**Hypochlorite:** A compound containing the hypochlorite ion ($OC1^-$), usually calcium or sodium hypochlorite.

**Hypochlorous Acid:** An acid formed by the addition of chlorine gas to water, HOC1.

**Inorganic:** Of mineral origin.

**Manifold:** A central pipe system providing multiple-source connections and/or feed points.

**Meter:** A device used to measure and indicate a physical value (flow, temperature, etc.).

**mgd:** Million gallons per day; rate of flow.

**Microorganism:** Extremely small plants or animals normally invisible to the naked eye.

**Open-Loop Control System:** A control system in which the control action is independent of the output.

**Organic Matter:** Chemical substances of animal or vegetable origin or, more accurately, of basically carbon structure, comprising hydrocarbons and their derivatives.

**Ordmtolidine Test (OT):** The determination of residual chlorine in water using orthotolidine reagent and calorimetric standards.

**Ortsotolidine Arsemite Test (OTA):** Similar to orthotolidine test; good for isolating interfering substance.

**Output:** The actual response obtained from the control system.

**Ozone:** A triatomic form of oxygen used as an oxidizing agent for disinfection; taste, color, or odor removal.

**Parts per Million (ppm):** The number of weight or volume units of a minor constituent present with each one million units of the major constituent of a solution or mixture. The results of most water and wastewater analyses were formerly expressed by ppm, but more recently the term has been replaced by the ratio of milligrams per liter.

**pH:** The reciprocal of the logarithm of the hydrogen-ion concentration. The concentration is the weight of hydrogen ions, in grams, per liter of solution. Neutral water, for example, has a pH value of 7 and a hydrogen-ion concentration of $10^{-7}$.

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**Postchlorination:** The application of chlorine to water subsequent to any treatment, including prechlorination.

**Potable Water:** Water that does not contain objectionable pollution, contamination, minerals, or infection and that is considered satisfactory for domestic consumption; in short, suitable for drinking purposes.

**Prechlorination:** The application of chlorine to water prior to any subsequent treatment.

**Primary Element:** A mechanism that will detect a physical change and give an output proportional to this change either mechanically, pneumatically, or electrically.

**Process Time Lag:** The delay of one condition with respect to another; for example, the interval between the mixing of chlorine gas with water and the satisfaction of chlorine demand.

**Proportional Position Control:** Generally called simple proportional control; one where there is a fixed linear relationship between the value of a measured variable (water flow rate) and the position of a final control element.

**Purification:** Removal of undesirable material from water.

**Range:** The difference between the smallest and greatest value of a variable.

**Range Adjustment:** A mechanical or electrical adjustment to fix the range of the instrument.

**Rechlorination:** The application of chlorine to water, at one or more points, following previous chlorination treatment.

**Recorder:** A device that records and indicates the measurement of a variable.

**Reproducibility:** The degree of closeness with which the same value of a variable can be measured at different times.

**Residual Chlorine:** See chlorine residual.

**Sodium Chlorite:** A chemical used in water treatment to produce chlorine dioxide, a gas; $NaC1O_2$.

**Sodium Hypochlorite:** A water solution of sodium hydroxide and chlorine in which sodium hypochlorite is the essential ingredient; NaOC1.

**Standard Methods:** Methods of analysis of water and wastewater published jointly by American Public Health Association, American Water Works Association, and Water Pollution Control Federation.

**Sterilization:** Destruction of all living organisms.

**Superchlorination:** The application of chlorine to water to provide free residual chlorination, in which the residual is usually so large as to require dechlorination.

**Titrator:** A device for the accurate measurement of an oxidizing agent.

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APPENDIX 3

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# الملحق الثالث

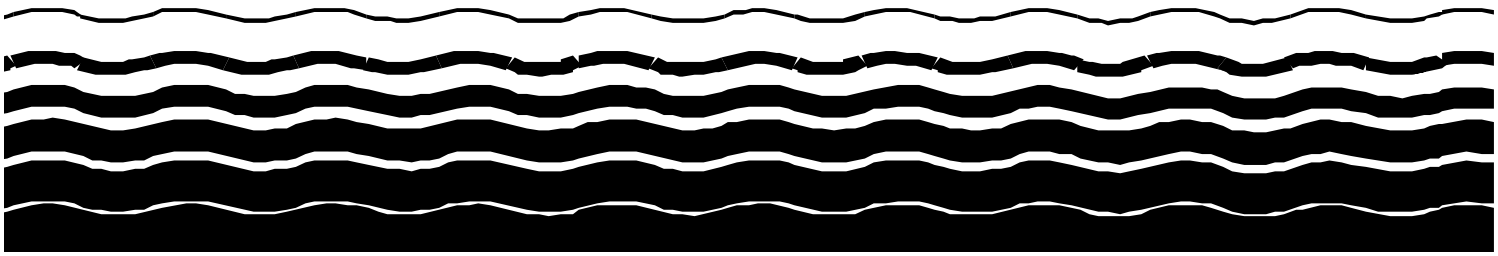
# Annex 3

United States
Environmental Protection
Agency

Office of Water
(4607)

EPA 815-R-99-014
April 1999

# Alternative Disinfectants and Oxidants Guidance Manual



# DISCLAIMER

This manual provides accurate technical data and engineering information on disinfectants and oxidants that are not as widely used as chlorine. The U.S. Environmental Protection Agency encourages drinking water treatment utilities and drinking water primacy agencies to examine all aspects of their current disinfection practices to improve the quality of their finished water without reducing microbial protection.

This document is EPA guidance only. It does not establish or affect legal rights or obligation. EPA decisions in any particular case will be made applying the laws and regulation on the basis of specific facts when permits are issued or regulations promulgated.

Mention of trade names or commercial products does not constitute an EPA endorsement or recommendation for use.

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# ACRONYMS

| | |
|---|---|
| AECL | Alternate enhanced coagulant level |
| ACUK | Acid chrome violet K |
| AOC | Assimilable organic carbon |
| ASDWA | Association of State Drinking Water Administrators |
| AWWA | American Water Works Association |
| AWWARF | AWWA Research Foundation |
| BAC | Biologically active carbon |
| BAF | Biologically active filtration |
| BAT | Best Available Technology |
| BCAA | Bromochloroacetic acid |
| BDOC | Biodegradable organic carbon |
| BMP | Best management practice |
| BOM | Biodegradable Organic Matter (=BDOC + AOC) |
| Br- | Bromide ion |
| BrO2- | Bromite ion |
| BrO3- | Bromate ion |
| CI | Confidence interval |
| Cl2 | Chlorine |
| ClO2 | Chlorine Dioxide |
| CT | Concentration-Time |
| CWS | Community Water System |
| D/DBP | Disinfectants/disinfection byproducts |
| DBPR | Disinfectants/disinfection byproducts rule |
| DBP | Disinfection byproduct |
| DBPFP | Disinfection byproduct formation potential |
| DBPP | Disinfection byproduct precursors |
| DBPRAM | DBP Regulatory Assessment Model |
| DBPs | Disinfection byproducts |
| DOC | Dissolved organic carbon |
| DPD | N,N-diethyl-p-phenylenediamine |
| DWEL | Drinking Water Equivalent Level |
| EBCT | Empty bed contact time |
| EMSL | EPA Environmental Monitoring and Support Laboratory (Cincinnati) |
| EPA | United States Environmental Protection Agency |
| ESWTR | Enhanced Surface Water Treatment Rule |
| FBR | Filter Backwash Rule |
| FY | Fiscal year |
| GAC | Granular activated carbon |
| GWR | Ground Water Rule |
| GWSS | Ground Water Supply Survey |
| H2O2 | Hydrogen Peroxide |
| HAA5 | Haloacetic acids (five) |
| HOBr | Hypobromous acid |
| HOCl | Hypochlorous acid |
| IC | Ion chromotography |

| | |
|---|---|
| ICR | Information Collection Rule |
| IESWTR | Interim Enhanced Surface Water Treatment Rule |
| IOA | International Ozone Association |
| IOC | Inorganic chemical |
| KMnO4 | Potassium permanganate |
| LOAEL | Lowest observed adverse effect level |
| LOQ | Limit of quantitation |
| LT1ESWTR | Long Term Stage 1 Enhanced Surface Water Treatment Rule |
| M-DBP | Microbial and disinfection byproducts |
| MCL | Maximum Contaminant Level |
| MCLG | Maximum Contaminant Level Goal |
| MDL | Method Detection Limit |
| mg/L | Milligrams per liter |
| mgd | Million gallons per day |
| MIB | Methylisoborneol |
| MRDL | Maximum Residual Disinfectant Level (as mg/l) |
| MRDLG | Maximum Residual Disinfectant Level Goal |
| MRL | Minimum Reporting Level |
| MX | 3-chloro-4-(dichloromethyl)-5-hydroxyl-2(5H)-furanone |
| NaCl | Sodium chloride |
| NCI | National Cancer Institute |
| ND | Not detected |
| NH2Cl | Monochloramine |
| NIPDWR | National Interim Primary Drinking Water Regulation |
| NIOSH | National Institute for Occupational Safety and Health |
| NOAEL | No Observed Adverse Effect Level |
| NOM | Natural Organic Matter |
| NOMS | National Organic Monitoring Survey |
| NORS | National Organics Reconnaissance Survey for Halogenated Organics |
| NPDWR | National Primary Drinking Water Regulation |
| NTNCWS | Nontransient noncommunity water system |
| NTP | Normal Temperature and Pressure |
| O2 | Oxygen |
| O3 | Ozone |
| OBr- | Hypobromite ion |
| OCl- | Hypochlorite ion |
| PCE | Perchloroethylene |
| PE | Performance evaluation |
| POE | Point-of-Entry Technologies |
| POU | Point-of-Use Technologies |
| ppb | Parts per billion |
| ppm | Parts per million |
| PQL | Practical Quantitation Level |
| PTA | Packed Tower Aeration |
| PWS | Public water system |
| RIA | Regulatory Impact Analysis |
| RMCL | Recommended Maximum Contaminant Level |
| RNDB | Regulations Negotiation Data Base |
| RSC | Relative Source Contribution |
| SDWA | Safe Drinking Water Act, or the “Act,” as amended in 1996 |
| SM | Standard Method |

| | |
|---|---|
| SMCL | Secondary Maximum Contaminant Level |
| SMR | Standardized mortality ratios |
| SOC | Synthetic Organic Chemical |
| SWTR | Surface Water Treatment Rule |
| TCE | Trichloroethylene |
| THM | Trihalomethane |
| THMFP | Trihalomethane formation potential |
| TMV | Tobacco mosaic virus |
| TOC | Total organic carbon |
| TTHM | Total trihalomethanes |
| TWG | Technologies Working Group |
| UV | Ultraviolet |
| VOC | Volatile Organic Chemical |
| WIDB | Water Industry Data Base |
| WS | Water supply |
| XDBPs | Halogenated DBPs |

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# 1. INTRODUCTION

## 1.1 Objective of this Manual

Chlorine is, by far, the most commonly used disinfectant in the drinking water treatment industry (Sawyer et al., 1994). Today, chlorine is used as a primary disinfectant in the vast majority of all surface water treatment plants, being used as a pre-disinfectant in more than 63 percent and as a post-disinfectant in more than 67 percent of all surface water treatment plants (USEPA, 1997). This manual is organized to provide technical data and engineering information on disinfectants that are not as widely used as chlorine. Also, where applicable, this document describes the use of these disinfectants as oxidants and any associated implications.

The U.S. Environmental Protection Agency (EPA) encourages utilities to examine all aspects of their current disinfection practices to identify opportunities to improve the quality of the finished water without reducing microbial protection. The objective of this guidance manual is to describe alternative disinfectants and disinfection techniques that may be used to comply with both the Stage 1 Disinfectants and Disinfection Byproducts Rule (DBPR) and Interim Enhanced Surface Water Treatment Rule (IESWTR) and highlight advantages and disadvantages of their use.

EPA is not recommending that utilities employ the disinfectants and oxidants discussed in this manual, nor is it advocating that utilities switch from one disinfectant or oxidant to another. EPA acknowledges that selection of the most appropriate disinfection technique is a site-specific decision best left to utility personnel and state agencies. Utilities should use this guidance as an information resource to assist in the selection of appropriate disinfectants and disinfectant schemes to meet their specific goals. Extensive bench and/or pilot scale testing and a thorough review of regulatory requirements should precede changes to disinfection practice. Systems should refer to the Guidance Manual for Compliance with the Filtration and Disinfection Requirements for Public Works Systems Using Surface Water Sources (AWWA, 1991) to ensure disinfectant schemes meet regulatory log inactivation requirements. Utilities should also refer to EPA's *Disinfection Profiling and Benchmarking Guidance Manual* (currently in production) to ensure compliance with the new regulatory requirements of the IESWTR.

This chapter presents a brief discussion of the background and regulatory context of alternative disinfectants, including an overview of the disinfection profiling and benchmarking approach to evaluate disinfection efficiency. In addition, a decision-making framework is provided that utilities can employ to assess the applicability of various disinfectants and disinfection strategies for individual systems. Chapter 2 presents an overview of disinfection, including the use of chlorine, with the next six chapters of this manual devoted to each of the following alternative disinfectants and oxidants:

- ***Chapter 3*** - Ozone ($O_3$);
- ***Chapter 4*** - Chlorine dioxide ($ClO_2$);
- ***Chapter 5*** - Potassium permanganate ($KMnO_4$);
- ***Chapter 6*** - Chloramine ($NH_2Cl$);
- ***Chapter 7*** - Ozone/hydrogen peroxide combinations ($O_3/H_2O_2$); and
- ***Chapter 8*** - Ultraviolet radiation (UV).

For each disinfectant, this guidance manual describes the chemistry specific to the disinfection or oxidation process, generation, primary uses and points of application, disinfection byproduct (DBP) formation, pathogen inactivation and disinfection efficacy, the status of analytical methods for residual monitoring, and operational considerations. Chapter 9 provides similar information regarding the use of combined disinfectants. A summary of existing disinfectant usage in the United States is provided in Appendix A. Cost estimates for the use of alternative disinfectants are provided in Appendix B.

## 1.2 Background

The most important use of disinfectants in water treatment is to limit waterborne disease and inactivate pathogenic organisms in water supplies. The first use of chlorine as a continuous process in water treatment was in a small town in Belgium in the early 1900s (White, 1992). Since introduction of filtration and disinfection at water treatment plants in the United States, waterborne diseases such as typhoid and cholera have been virtually eliminated. For example, in Niagara Falls, NY between 1911 and 1915, the number of typhoid cases dropped from 185 deaths per 100,000 population to nearly zero following introduction of filtration and chlorination (White, 1986).

In 1974, researchers in the Netherlands and the United States demonstrated that trihalomethanes (THMs) are formed as a result of drinking water chlorination (Rook, 1974; Bellar et al., 1974). THMs are formed when chlorine or bromide reacts with organic compounds in the water. EPA subsequently conducted surveys confirming widespread occurrence of THMs in chlorinated water supplies in the United States (Symons et al., 1975; USEPA, 1978). THMs and other DBPs have been shown to be carcinogenic, mutagenic, etc. These health risks may be small, but with the large population exposed, need to be taken seriously.

As a result of DBP concerns from chlorine, EPA, as well as the water treatment industry, placed more emphasis on the use of disinfectants other than chlorine. Some of these alternative disinfectants, however, have also been found to produce DBPs as a result of either reactions between disinfectants and compounds in the water or as a natural decay product of the disinfectant itself (McGuire et al., 1990; Legube et al; 1989). These DBPs include:

- Halogenated organics, such as THMs, haloacetic acids, haloketones, and others, that are produced primarily as a result of chlorination.

- Organic oxidation byproducts such as aldehydes, ketones, assimilable organic carbon (AOC), and biodegradable organic carbon (BDOC), that are associated primarily with strong oxidants such as ozone, chlorine, and advanced oxidation; and
- Inorganics such as chlorate and chlorite, associated with chlorine dioxide, and bromate, that is associated with ozone, and has also has been found when chlorine dioxide is exposed to sunlight.

As documented in this manual, the type and amount of DBPs produced during treatment depends largely on disinfectant type, water quality, treatment sequences, contact time, and environmental factors such as temperature and pH.

When considering the use of alternative disinfectants, systems should ensure that the inactivation of pathogenic organisms is not compromised. Pathogens pose an immediate critical public health threat due to the risk of an acute disease outbreak. Although most identified public health risks associated with DBPs are chronic, long-term risks, many systems will be able to lower DBP levels without compromising microbial protection.

## 1.3 Regulatory Context

Pursuant to requirements of the Safe Drinking Water Act (SDWA) Amendments of 1996, EPA is developing interrelated regulations to control microbial pathogens and disinfectant residuals and disinfection byproducts in drinking water. These rules are collectively known as the microbial/disinfection byproducts (M-DBP) rules and are intended to address complex risk trade-offs between the desire to inactivate pathogens found in water and the need to reduce chemical compounds formed as byproducts during disinfection. The rules are being promulgated in two phases.

As part of the first phase, the Stage 1 DBPR and the IESWTR were promulgated on December 16, 1998 (63 FR 69390 and 63 FR 69478, respectively). The Stage 1 DBPR applies to all community water systems (CWS) and non-transient, non-community water systems (NTNCWS) that treat their water with a disinfectant for either primary or residual treatment. The IESWTR amends the Surface Water Treatment Rule (SWTR) and includes new and more stringent requirements to control waterborne pathogens including specifically the protozoan *Cryptosporidium*. The IESWTR applies to all public water systems that use surface water, or ground water under the direct influence of surface water as defined at 40 CFR, Part 141, Subpart H[1], and that serve at least 10,000 people.

Three future rules, also included in phase one include the Long-Term 1 Enhanced Surface Water Treatment Rule (LT1ESWTR), the Ground Water Rule (GWR), and the Filter Backwash Rule (FBR). Each of these rules are planned for promulgation in November 2000. The LT1ESWTR will address pathogen inactivation and removal requirements for Subpart H systems serving fewer than 10,000 people. The GWR will specify appropriate disinfection techniques, including the use of best

[1] Subpart H systems are defined as public water systems supplied by a surface water source or by a ground water source under the direct influence of surface water.

management practices (BMPs) and source control measures. The FBR will set a standard for filter backwash recycling for all public water systems regardless of size.

The second phase, consisting of the Stage 2 DBPR and the Long-Term 2 ESWTR, will be promulgated in the year 2002 and will revisit the regulations for the formation of DBPs for all systems and the inactivation and removal of pathogens for surface water systems, respectively.

The projected dates for future M-DBP regulatory activities are summarized in Table 1-1.

**Table 1-1. Key Dates for Regulatory Activities**

| Date | Regulatory Action |
|---|---|
| November 2000 | Promulgate Long-Term 1 Enhanced Surface Water Treatment Rule |
| November 2000 | Promulgate Ground Water Rule |
| November 2000 | Promulgate Filter Backwash Rule |
| May 2002 | Promulgate Stage 2 Disinfectants and Disinfection Byproduct Rule |
| May 2002 | Promulgate Long-Term 2 Enhanced Surface Water Treatment Rule |

Concurrent with the M-DBP rules, in May 1996, EPA promulgated the Information Collection Rule (ICR) to obtain data on source water quality, byproduct formation, and drinking water treatment plant design and operations. The ICR applies to Subpart H systems serving more than 100,000 people and ground water systems serving more than 50,000 people. EPA intended to use data from the ICR to address completely the complex trade-offs between chronic DBP health risks and acute pathogenic health risks, but delays in promulgation of the ICR eliminated this potential data source for use in the IESWTR. Until the ICR data are analyzed in detail, EPA cannot fully address the issue of DBP and pathogenic risk trade-offs.

National Primary and Secondary Drinking Water Regulations, published in 40 CFR Parts 141 and 143, respectively, limit the amount of specific contaminants and residual disinfectants and classes of these compounds that are delivered to users of public water systems. These limits are expressed as follows:

- **Maximum Contaminant Level Goals (MCLGs).** MCLGs are non-enforceable health goals for public water systems. MCLGs are set at levels that, in the EPA Administrator's judgment, allow no known or anticipated adverse effect on the health of persons to occur and that allow an adequate margin of safety.
- **Maximum Residual Disinfectant Level Goals (MRDLGs).** As with MCLGs, EPA has established MRDLGs for disinfectants at levels at which no known or anticipated adverse effects on the health of persons occur and that allow an adequate margin of safety. MRDLGs are non-enforceable health goals based only on health effects and exposure information and do not reflect the benefit of the addition of the chemicals for control of waterborne microbial contaminants.

- **Maximum Contaminant Levels (MCLs).** MCLs are enforceable standards set as close to the MCLGs as technically and economically feasible.
- **Maximum Residual Disinfectant Levels (MRDLs).** MRDLs are similar to MCLs. MRDLs are enforceable standards, analogous to MCLs, that recognize the benefits of adding a disinfectant to water on a continuous basis and of addressing emergency situations such as distribution system pipe ruptures. As with MCLs, EPA has set the MRDLs as close to the MRDLGs as feasible.

In November 1979, the EPA set an interim MCL for Total THMs (TTHMs) of 0.10 mg/L as an annual average for systems serving at least 10,000 people. This standard was based on the need to reduce THM levels due to suspected carcinogenicity. Since then, EPA has developed and promulgated standards for numerous contaminants. As of the December 16, 1998 DBPR promulgation, MCLGs, MCLs, MRDLGs, and MRDLs are as presented in Tables 1-2 through 1-4. As included in these tables, the December 16, 1998 Stage 1 DBPR:

- Lowered the existing MCL for TTHMs from 0.10 mg/L to 0.080 mg/L;
- Extended the MCL for TTHMs to all size systems;
- Requires enhanced coagulation or enhanced precipitative softening for certain systems;
- Established MRDLs and MRDLGs for chlorine, chloramine, and chlorine dioxide;
- Established MCLs for haloacetic acid (five) (HAA5), bromate, and chlorite, and
- Established MCLGs for eight disinfection byproducts.

Further, the SWTR of 1989 requires 3.0-log inactivation for *Giardia* cysts and 4.0-log inactivation for viruses in surface water supplies. To meet these goals, the SWTR established treatment requirements for filtration and disinfection. As shown in Table 1-5, these goals can be met using various treatment schemes that include filtration and disinfection. The IESWTR requires a 2.0 log removal of *Cryptosporidium* for Subpart H systems serving at least 10,000 people.

**Table 1-2. Primary Drinking Water Regulations Related to Microbiological Contaminants**

| Compound | MCLG (mg/L) | MCL (mg/L) | Potential Health Effects | Sources of Drinking Water Contamination |
|---|---|---|---|---|
| *Giardia lamblia* | Zero | TT[1] | Gastroenteric disease | Human and animal fecal waste |
| *Legionella* | Zero | TT | Legionnaire's disease | Common bacteria in natural waters; can proliferate in water heating systems |
| Heterotrophic Plate Count | N/A | TT | Indicates water quality, effectiveness of treatment | |
| Total Coliform | Zero | < 5.0%[2] | Indicates potential presence of gastroenteric pathogens | Human and animal fecal waste |
| Turbidity | N/A | TT | Indicates water treatment failure and pathogens in drinking water | Particles from storm runoff, discharges into source water, and erosion |
| Viruses | Zero | TT | Gastroenteric disease | Human and animal fecal waste |

Source: AWWA Internet, 1997.

[1] TT = Treatment technique requirement in lieu of MCL as established in 40 CFR §141.70.

[2] No more than 5.0 percent positive if >40 samples/month. No more than 1 positive if <40 samples/month [40 CFR §141.63(a) ].

**Table 1-3. Primary Drinking Water Regulations Related to Disinfection Byproducts**

| Compound | MCLG (mg/L) | MCL (mg/L) | Potential Health Effects | Sources of Drinking Water Contamination |
|---|---|---|---|---|
| Bromate | Zero[3] | 0.010[4] | Cancer | Ozonation byproduct |
| Bromodichloromethane | Zero[3] | see TTHMs | Cancer, liver, kidney, and reproductive effects | Drinking water chlorination and chloramination byproduct |
| Bromoform | Zero[3] | see TTHMs | Cancer, nervous system, liver and kidney effects | Drinking water ozonation, chloramination, and chlorination byproduct |
| Chlorite | 0.8[3] | 1.0[4] | Hemolytic anemia | Chlorine dioxide disinfection byproduct |
| Chloroform | Zero[3] | see TTHMs | Cancer, liver, kidney, reproductive effects | Drinking water chlorination and chloramination byproduct |
| Dibromochloromethane | 0.06[3] | see TTHMs | Nervous system, liver, kidney, reproductive effects | Drinking water chlorination and chloramination byproduct |
| Dichloroacetic Acid | Zero[3] | See HAA5 | Cancer and other effects | Drinking water chlorination and chloramination byproduct |
| Haloacetic Acids[1] (HAA5) | N/A | 0.060[4] | Cancer and other effects | Drinking water chlorination and chloramination byproduct |
| Trichloroacetic Acid | 0.3[3] | See HAA5 | Possibly cancer and reproductive effects | Drinking water chlorination and chloramination byproduct |
| Total Trihalomethanes[2] (TTHMs) | N/A | 0.08[4] | Cancer and other effects | Drinking water chlorination and chloramination byproduct |

Source: 63 *FR* 69390 (12/16/98)

[1] HAA5 is the sum of the concentrations of mono-, di-, and trichloroacetic acids and mono-and dibromoacetic acids.

[2] Total Trihalomethanes are the sum of the concentrations of bromodichloromethane, dibromochloromethane, bromoform, and chloroform.

[3] Finalized on December 16, 1998 (63 FR 69390) as established in 40 CFR §141.53.

[4] Finalized on December 16, 1998 (63 FR 69390) as established in 40 CFR §141.64

**Table 1-4. Primary Drinking Water Regulations Related to Residual Disinfectants**

| Disinfectant | MRDLG[3] (mg/L) | MRDL[4] (mg/L) |
|---|---|---|
| Chlorine[1] | 4 (as $Cl_2$) | 4.0 (as $Cl_2$) |
| Chloramine[2] | 4 (as $Cl_2$) | 4.0 (as $Cl_2$) |
| Chlorine Dioxide | 0.8 (as $ClO_2$) | 0.8 (as $ClO_2$) |

[1] Measured as free chlorine
[2] Measured as total chlorine
[3] Finalized on December 16, 1998 (63 *FR* 69390) as established in 40 CFR §141.54.
[4] Finalized on December 16, 1998 (63 *FR* 69390) as established in 40 CFR §141.65

**Table 1-5. Log Removal/Inactivation through Filtration and Disinfection Required Under the SWTR**

| Process | *Giardia* cysts | Virus |
|---|---|---|
| ***Total log removal/inactivation Required*** | ***3.0*** | ***4.0*** |
| Conventional sedimentation/filtration credit | 2.5 | 2.0 |
| Disinfection inactivation required | 0.5 | 2.0 |
| Direct filtration credit | 2.0 | 1.0 |
| Disinfection inactivation required | 1.0 | 3.0 |
| Slow sand filtration credit | 2.0 | 2.0 |
| Disinfection inactivation required | 1.0 | 2.0 |
| Diatomaceous earth credit | 2.0 | 1.0 |
| Disinfection inactivation required | 1.0 | 3.0 |
| No Filtration | 0.0 | 0.0 |
| Disinfection inactivation required | 3.0 | 4.0 |

Source: AWWA, 1991.
Note: Some instances may require higher than 3 and 4 log removal. Also, some states may reduce removal filtration process.

## 1.3.1 Disinfection Profiling and Benchmarking

The IESWTR establishes disinfection benchmarking as a procedure requiring certain PWSs to evaluate the impact on microbial risk of proposed changes in disinfection practice. It is designed to facilitate utilities and States working together to assure that pathogen control is maintained while the provisions of the Stage 1 DBPR are implemented. This procedure involves a PWS charting daily levels of pathogen inactivation for a period of at least one year to create a profile of inactivation performance. The PWS then uses this profile to determine a baseline or benchmark of inactivation against which proposed changes in disinfection practices can be measured.

Systems are required to prepare a disinfection profile if either TTHM or HAA5 levels are at least 0.064 or 0.048 mg/L, respectively, as an annual average. These levels, equal to 80 percent of the MCLs established for these compounds by the Stage 1 DBPR, are intended to include most systems that will modify their disinfection practices to comply with the Stage 1 DBPR. To determine applicability, systems that collected TTHM and HAA5 data under the ICR must use the results of the

last 12 months of ICR monitoring unless the State determines there is a more representative data set. Non-ICR systems may use existing TTHM and HAA5 data, if approved by the State, or must conduct TTHM and HAA5 monitoring for four quarters. This monitoring must be completed no later than 15 months after promulgation of the IESWTR (i.e., by March, 2000). Alternatively, systems can elect to forgo this monitoring if they construct a disinfection profile.

A disinfection profile consists of a compilation of daily *Giardia lamblia* log inactivations (plus virus inactivations for systems using either chloramines or ozone for primary disinfection) computed over a period of at least one year. It is based on daily measurements of disinfectant residual concentration(s), contact time(s), temperature, and pH. The profile may be developed using up to 3 years of existing (i.e. grandfathered) data, if the State finds the data acceptable. Systems having less than 3 years of acceptable grandfathered data are required to conduct one year of monitoring to create the profile. This monitoring must be complete within 27 months of IESWTR promulgation (i.e., by March, 2001). The disinfection benchmark is equal to the lowest monthly average inactivation level in the disinfection profile (or average of low months for multi-year profiles).

Any system required to develop a disinfection profile under the IESWTR that decides to make a significant change to its disinfection practice must consult with the State prior to making the change. Significant changes in disinfection practice are defined as: 1) moving the point of disinfection, not including routine seasonal changes, 2) changing the type of disinfectant, 3) changing the disinfection process, and 4) other modifications designated as significant by the State. As part of the consultation process, the system must submit to the State the following information: a description of the proposed change; the disinfection profile for Giardia lamblia (and, if necessary, viruses) and benchmark; and an analysis of how the proposed change will affect the current levels of disinfection. In addition, the State is required to review the disinfection profile a part of its periodic sanitary survey.

For more information on disinfection profiling and benchmarking, refer to EPA's *Disinfection Profiling and Benchmarking Guidance Manual* (expected to be available in 1999).

## 1.4 Use of Disinfectants as Chemical Oxidants

Most disinfectants are strong oxidants and/or generate oxidants as byproducts (such as hydroxyl free radicals) that react with organic and inorganic compounds in water. While the primary focus of this manual is disinfection, many of the disinfectants described in this manual are also used for other purposes in drinking water treatment, such as taste and odor control, improved flocculation, and nuisance control. Because DBPs are produced irrespective of the intended purpose of the oxidant, it is important to also address uses of disinfectants as oxidants in water treatment. These additional uses are described in more detail in Chapter 2.

## 1.5 How Chlorine is Addressed in this Guidance Manual

This guidance manual does not provide as broad a discussion of chlorine and chlorination practices as it does for the capabilities and uses of alternative disinfectants. There are two reasons EPA has

taken this approach: 1) the goal of this manual is to provide technical and engineering information to utilities and the states on alternative disinfectants and oxidants, about which there is less comprehensive information than for chlorine; and 2) a great majority of utilities already use chlorine, in a wide variety of applications, for which there exists a wealth of literature on chlorine's uses and performance capabilities. Summarizing this large body of knowledge in this guidance is neither practical nor necessary.

This manual is not an EPA endorsement of alternative disinfectants nor is it a recommendation for utilities to switch from chlorine to an alternative disinfectant. Rather, this manual provides technical and engineering information to assist local professionals in making treatment decisions. EPA believes that utility and state program personnel are best able to select disinfectants and design a disinfection scheme, based upon site-specific conditions, that meets operational and regulatory constraints. EPA does not require the use of chlorine or any other specific disinfectant for site-specific uses. Again, local professionals are best suited to select disinfectants to address the unique water treatment challenges posed by their source water and plant infrastructure.

EPA recognizes that, at the present time, chlorination is an important and central component of most water treatment regimes in this country. As such, a summary of the uses and capabilities of chlorine is provided in Section 2.7 of this manual. Section 2.7 also contains an extensive reference list for additional information on chlorination.

## 1.6 A Summary of Alternative Disinfectant Properties

Subsequent chapters in this manual discuss several disinfectant alternatives available to a water supplier. Table 1-6 summarizes the key technical and regulatory considerations associated with the use of the various disinfectants for selecting the most appropriate disinfectant. The table provides some broad guidelines to provide a framework for decision making. The ratings in Table 1-6 are based on a typical disinfectant application. Thus, even though chlorine is considered to be prone to THM formation, the table does not address the degree or amount of THMs produced. Similarly, more than 2-log inactivation can be achieved for some disinfectants, but the high dose required may not make it a reasonable application, and in that case, would be identified in the table as not able to achieve 2-log inactivation.

The following key issues are addressed in Table 1-6:

- **THMs, oxidized organics, and halogenated organics are produced.** Halogenated organics are formed when chlorine or ozone (in the presence bromide ion) is used, while oxidized organics occur in the greatest concentration when a strong oxidant is used. The production of DBPs depends on the amounts and types of precursors in the water.
- **Inorganic byproducts are produced.** Inorganic byproducts include chlorate ion, chlorite ion, and bromate ion associated with chlorine dioxide and ozone.

- **MRDLs are required for some disinfectants.** Note that this requirement must be balanced with the requirement to maintain a residual in the distribution system. For most disinfectants, such as chlorine, the MRDL is relatively high and will generally not create a problem.
- **Lime softening impacts are noted.** The high pH treatment during lime softening has an impact on chlorine, chloramines, and UV.
- **Turbidity impacts UV disinfection and ozonation.** Ozone may interfere with coagulation and settling. As such, it is recommended that ozonation be placed after settling but before filters to minimize turbidity impacts.
- **Inactivation requirements are divided into those achieving more or less than 2-log inactivation.** This differentiation is to identify the feasibility of achieving high inactivation at modest doses. For example, while chlorine can achieve 3-log *Giardia* cyst inactivation, the CT requirement for 3-log inactivation of 100 to more than 300 mg-min/L will require high chlorine doses and/or long contact times. However, 4-log virus inactivation is achievable with a CT of 15 to 60 mg·min/L for most temperatures.

  Recently there has been some reports of 2-log and higher *Cryptosporidium* oocyst inactivation with UV, using a system that concentrates the oocysts on a filter and then allows extended exposure to UV to doses as high as 8,000 $mW \cdot s/cm^2$. This type application is considered experimental at this time.
- **Applicability as a secondary disinfectant represents the ability of the disinfectant to maintain a residual in the distribution system.** Only chlorine, chlorine dioxide, and monochloramine provide residual disinfection in the distribution system. Chlorine dioxide is limited to systems with smaller distribution systems because the total chlorine dioxide dose that can be applied is limited by the production of chlorate ion and chlorite ion.
- **Operator skill provides general guidance to the amount of operator attention and maintenance required.** All of the disinfectants can be placed on automatic control to limit the amount of operator attention. The operational attention is rated based on the complexity of the disinfectant application. Therefore, permanganate, which is a simple chemical feed system with few mechanical elements, is rated 1 (low attention), while peroxone which include both ozone systems and hydrogen peroxide feed systems, is rated 5 (high attention).
- **All chemical disinfectants are judged to be applicable to small and large utilities.** Modular units of the technologies can cover a large range of flows. Ozone and chlorine dioxide generators are available with small and large capacities. Chlorine and chemical feed systems have been used successfully in all applications. Most UV water treatment facilities are less than 200 gpm in capacity.

Key elements in the decision making process relate to the water source (i.e., ground or surface water) and existing treatment configuration (including filtration) because these factors have a significant impact on the degree of log removal required during disinfection. Water quality has a large impact on the potential for DBP BOM formation.

**Table 1-6. Summary of Disinfectant Properties**

**(Based on Typical Disinfectant Application)**

| Condition | Chlorine | Ozone | Chlorine Dioxide | Permanganate | Chloramine | Ozone/Peroxide | Ultraviolet |
|---|---|---|---|---|---|---|---|
| Produce THM with TOC | y | s | n | n | y | s | n |
| Produce oxidized organics | s | y | s | s | n | y | s |
| Produce halogenated organics | y | s | n | n | y | s | n |
| Produce inorganic byproducts | n | s | y | n | n | s | n |
| Produce BOM | s | y | s | n | n | y | n |
| MRDL applies | y | n | y | n | y | n | n |
| Lime softening impacts | y | n | n | n | y | n | y |
| Turbidity impacts | n | s | n | n | n | s | y |
| Meet *Giardia* - <2.0 log | y | y | y | n | n | n | n |
| Meet *Giardia* - >2.0 log | n | y | y | n | n | n | n |
| Meet *Crypto* - <2.0 log | n | y | y | n | n | n | n |
| Meet *Crypto* - >2.0 log | n | y | n | n | n | n | n |
| Meet Virus - <2.0 log | y | y | y | n | n | n | y |
| Meet Virus - >2.0 log | y | y | y | n | n | n | y |
| Secondary disinfectant | y | n | s | n | y | n | n |
| Operator skill (1=low; 5=high) | 1 | 5 | 5 | 1 | 2 | 5 | 3 |
| Applicable to large utilities | y | y | y | y | y | y | n |
| Applicable to small utilities | y | y | y | y | y | y | y |

y = yes, n = no, s = sometimes

The following sections describe each of these phases.

# 1.7 Selecting a Disinfection Strategy

This section presents general guidance that can be used to assess the applicability of various disinfectants or combination of disinfectants to select an appropriate disinfection strategy. Because the selection of an appropriate strategy depends on site-specific conditions unique to each water supply system, final selection of a strategy should be made with appropriate technical guidance (e.g., engineering study/evaluation or bench or pilot scale testing of alternatives). Selecting the most appropriate disinfectant strategy for water treatment requires a balance among three key driving forces:

- **Providing water free of pathogens.** Since the SWTR, the regulatory focus for pathogen removal focused on coliform bacteria, heterotrophic plate counts, *Giardia* cysts, *Legionella*, and viruses. Recently, the focus has been expanded to include *Cryptosporidium* oocyst removal and inactivation, especially due to its resistance to chlorine.

- **Avoiding the production of disinfection byproducts (DBPs).** Trihalomethanes (THMs), other halogenated organics, ozone DBPs, oxidation byproducts, and some disinfectant residuals present a health risk and must be limited in drinking waters. DBP precursor removal through process optimization or enhanced coagulation is the first step in DBP control.
- **Requiring residual disinfectant to maintain the bacteriological quality in the water as it is distributed to customers to control regrowth.** The potential for DBP formation increases with extended contact between DBP precursors and residual disinfectants.

When changing disinfectants or oxidants water providers should consult with their primacy agencies. The impact of the change should consider the impact on disinfection credit using disinfection profiling and benchmarking techniques as summarized in Section 1.3.

## 1.7.1 Disinfection Strategy Evaluation

The selection of a disinfection strategy, as presented below, is affected by the following:

- Effectiveness of the current disinfection system;
- Need to change disinfectants;
- Selection of an alternative disinfectant; and
- Primary and secondary disinfection requirements.

As used in this guidance, primary and secondary disinfection are defined as follows:

**Primary disinfection:** The first (i.e., primary) disinfectant used in a treatment system, with the primary objective of the disinfectant being to achieve the necessary CT (i.e., microbial inactivation).

**Secondary disinfection:** The second disinfectant used in a treatment system, with the primary objective of the disinfectant being to maintain the disinfection residual through the distribution system.

For discussion, the approach to select a disinfection strategy is divided into three phases:

- Evaluate the current primary disinfection practice
- Select a primary disinfectant
- Select a secondary disinfectant.

### *1.7.1.1* Evaluate the Current Primary Disinfection Practice

Figure 1-1 presents the decision making process used to determine whether the present primary disinfectant can meet disinfection and byproduct requirements. The key decision points in Figure 1-1 include:

- **Meet current microbial inactivation limits.** Microbial limits are defined by the primary drinking water standards shown in Table 1-2. The regulated indicators of pathogens include

*Giardia lamblia*, *Legionella*, HPC, total coliform turbidity, and viruses. The disinfectant must be capable of meeting the inactivation requirements for disinfection. If not, the plant must determine if the current disinfectant can meet the microbial inactivation requirements solely through process modifications. A process modification may be to move the application point, increase dose, increase contact time, or adjust pH. If not, a new disinfectant may be needed. In some instances, a PWS may opt to improve its current microbial inactivation even though the PWS is meeting current microbial limitations. In these instances, an evaluation is necessary to ensure that any process modifications to improve inactivation will still provide compliance with the microbial limits.

- **Meet current DBP limits.** A second set of limits imposed on disinfectant usage are DBP requirements. The DBP limits are established in the Stage 1 DBP rule (See Table 1-3 and Table 1-5). To meet these limits on a consistent basis under normal varying water quality conditions, 80 percent of the MCL serves as an action level that requires a change in treatment practice. Similar to microbial inactivation, some PWSs may desire to improve current DBP protection, thus requiring evaluation of these modifications. By optimizing existing treatment processes, the production of DBPs can be reduced. Optimization may include pretreatment optimization (i.e., coagulation, filtration, etc.) or process modifications such as moving the point of disinfection. Enhanced coagulation is required by the Stage 1 DBP rule. Where process modifications are contemplated, the PWS must ensure that both microbial inactivation and DBP formation comply with applicable regulatory requirements. If optimized treatment cannot meet DBP and microbial requirements, a new disinfectant may be needed.

#### *1.7.1.2* Select a Primary Disinfectant

If it is determined that a new disinfectant is required or desired for better public health protection, the second phase in the decision process (Figure 1-2) addresses the factors concerning selection of a primary disinfectant. This decision requires knowledge of the following three key components:

- **TOC concentration.** A high TOC concentration indicates a high potential for DBP formation. In these cases, the decision tree will favor those disinfectants that will not produce DBPs or will produce the least amount of DBPs. Note that precursor removal and enhanced coagulation are used to reduce TOC during treatment optimization as indicated in Figure 1-1. "High TOC" quantifies the potential to produce DBPs and is defined as a condition meeting one of the following criteria:
  - TOC exceeds 2 mg/L;
  - TTHM exceeds MCL (0.08 mg/L under Stage 1 DBPR); or
  - HAA5 exceeds MCL (0.06 mg/L under Stage 1 DBPR).

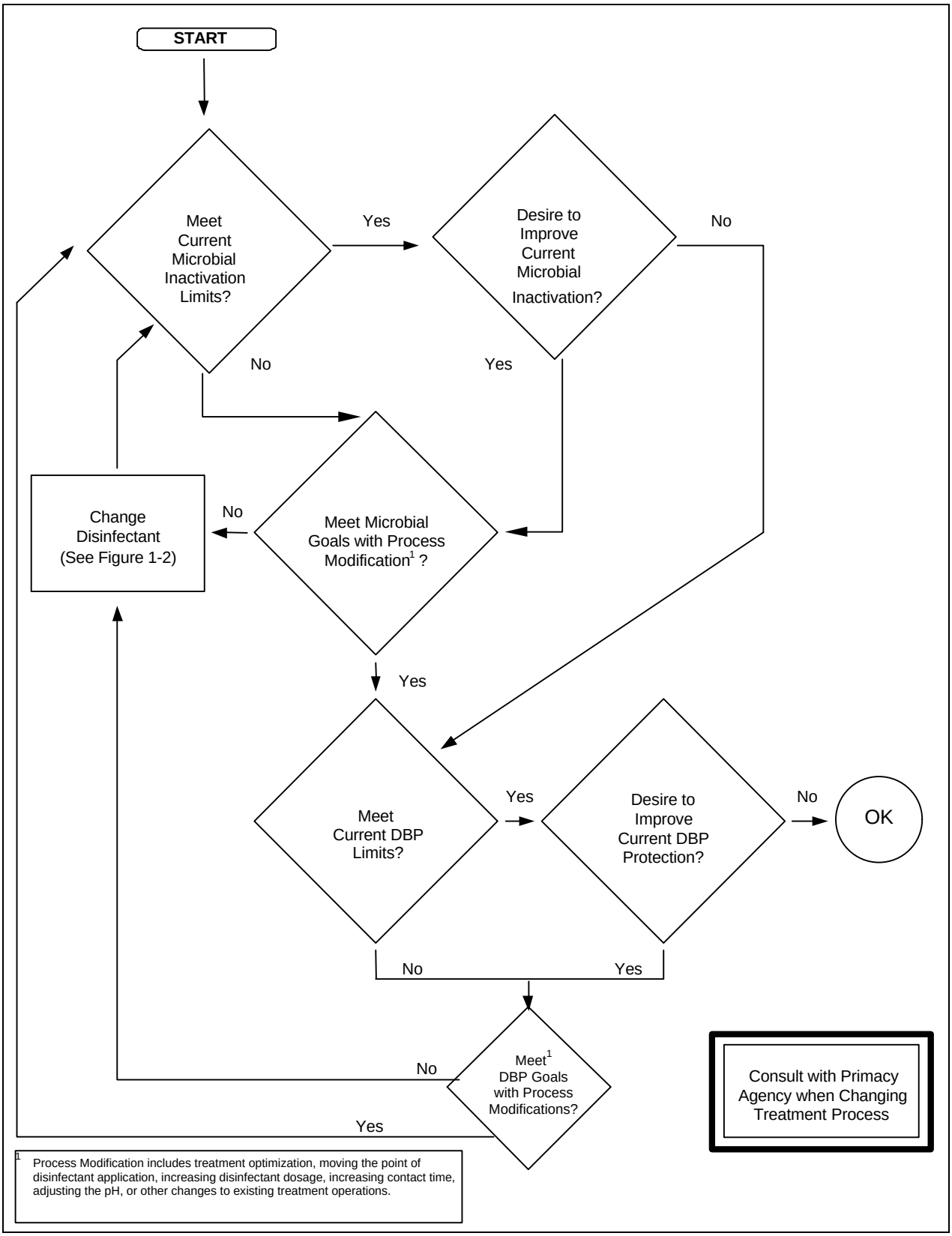


**Figure 1-1. Flow Diagram to Evaluate Current Disinfection Practices**

- **Bromide ion concentration.** The reactions of strong oxidants (ozone and peroxone) with bromide ion to produce hypobromous acid and bromate ion, precludes their usage with waters containing high concentrations of bromide ion. High bromide ion is defined as concentrations exceeding 0.10 mg/L.
- **Filtered versus non-filtered systems.** The use of ozone or ozone/peroxide for unfiltered systems without the benefit of biofiltration to reduce ozonation byproducts and BOM is strongly discouraged.

#### *1.7.1.3* Select a Secondary Disinfectant

The selection of a secondary disinfectant depends on the selected primary disinfectant. Figure 1-3 identifies three decision points for secondary disinfectants:

- **Assimilable organic carbon (AOC) concentration.** AOC is produced when a strong oxidant (e.g., ozone) is used as primary disinfectant in the presence of high TOC water. High AOC is defined as concentrations exceeding 0.10 mg/L after filtration. In these cases, additional biological or GAC treatment should be considered to stabilize the finished water and prevent regrowth in the distribution system.
- **DBP formation potential (DBPFP).** The DBPFP serves as an indication of the amount of organic byproducts that could be expected to form in the distribution system if chlorine is used. Because DBP formation continues in the distribution system, the DBP content at the plant effluent should be limited. A high DBPFP is defined as a water meeting one of the following criteria:
  - TTHM seven-day formation exceeds the MCL (0.08 mg/L under Stage 1 DBPR); or
  - HAA5 seven-day formation exceeds the MCL (0.06 mg/L under the Stage 1 DBPR).
- **Distribution system retention time.** In a large distribution system, booster stations may be required to maintain the disinfection residual. Since chlorine dioxide has an upper limit for application, its usage may not be feasible if relatively high doses are required to maintain a residual in the distribution system. A distribution system retention time is considered high if it exceeds 48 hours.

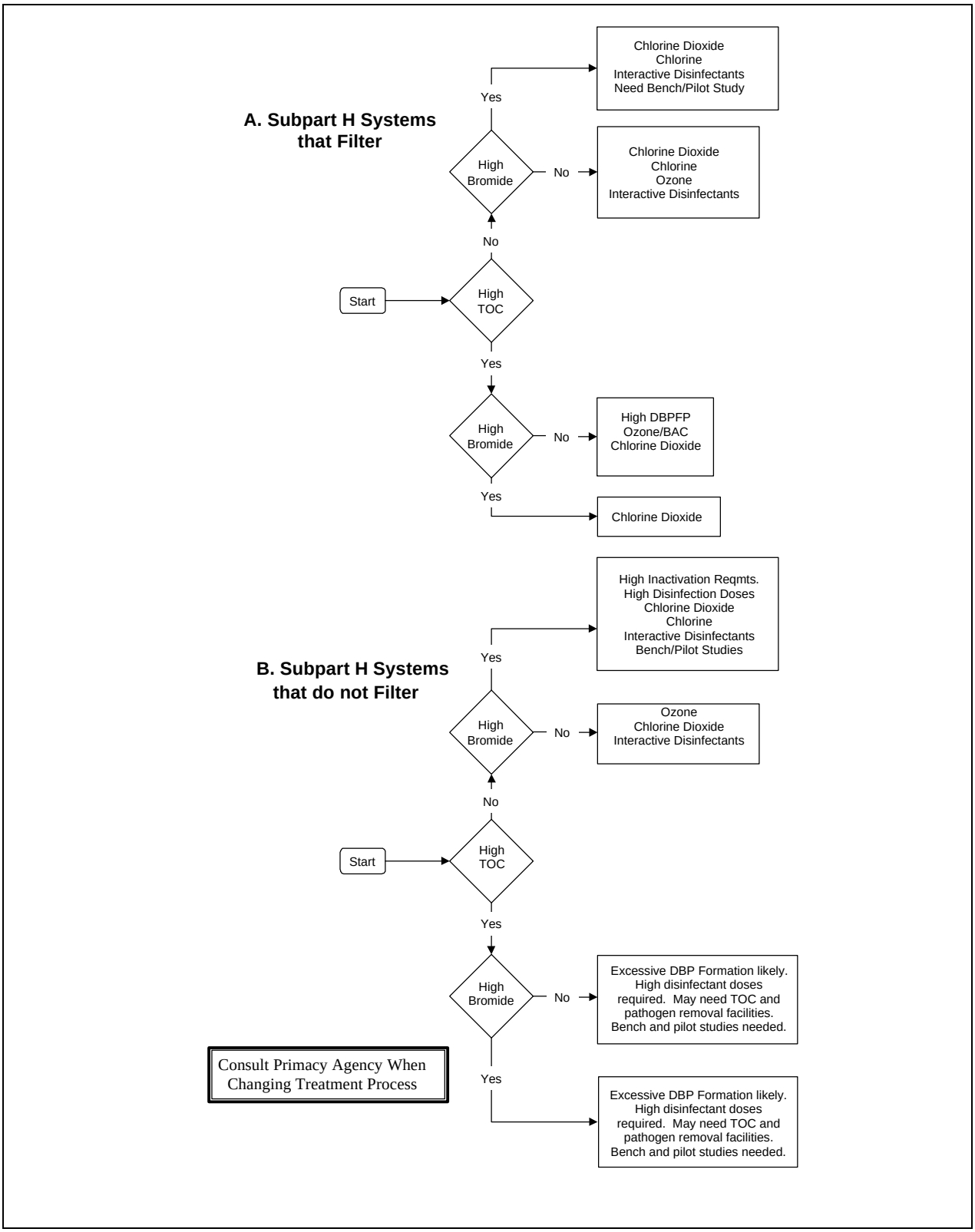


**Figure 1-2. Flow Diagram to Narrow Selection of a New Primary Disinfectant**

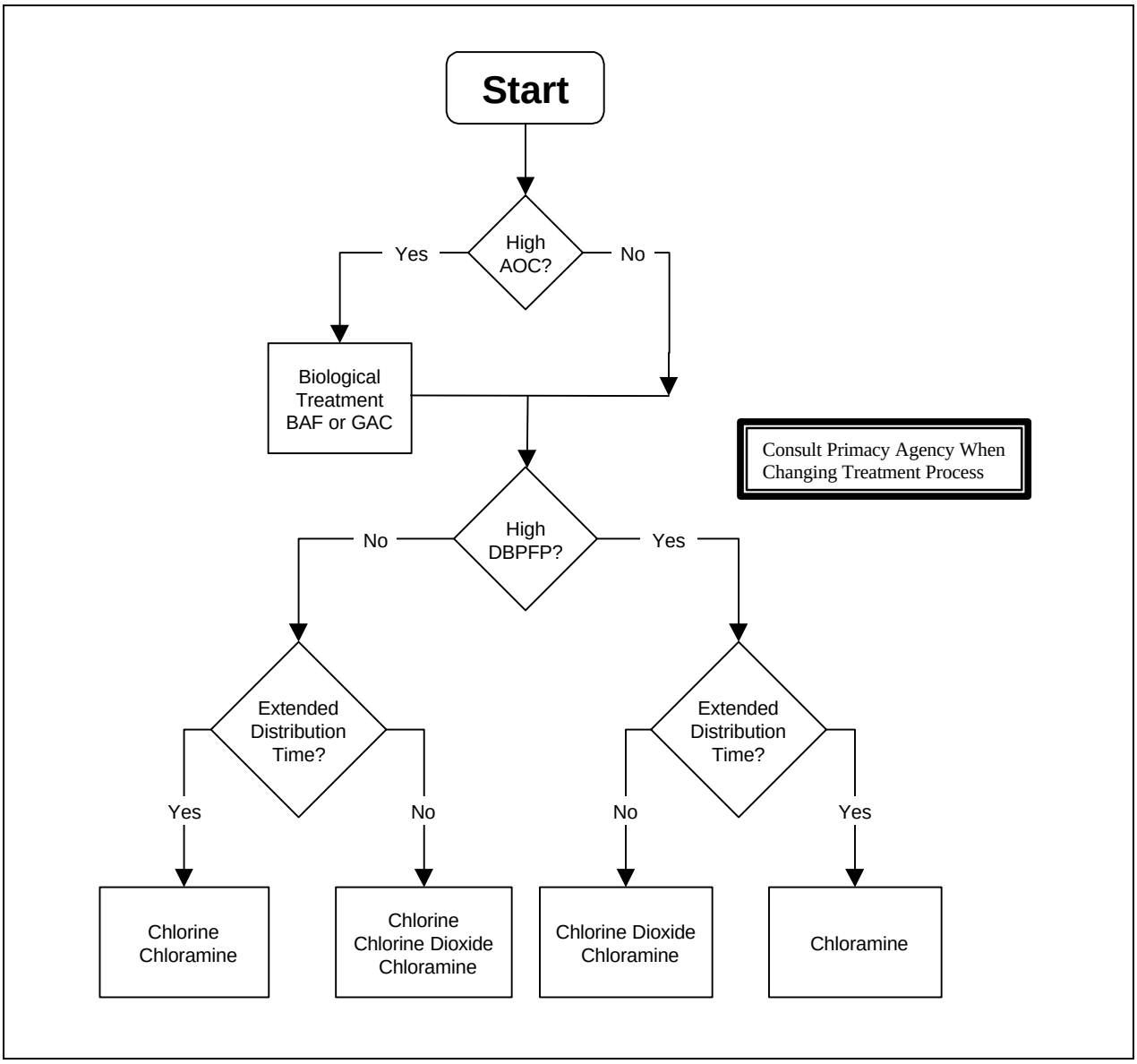


**Figure 1-3. Flow Diagram to Narrow Selection of a New Secondary Disinfectant**

## 1.7.2 Summary

The approach outlined above serves as a basis to evaluate the need for and to select the most appropriate alternative disinfectants for drinking water systems. The approach is general enough to cover the likely outcomes of the inactivation requirements and DBP formation. However, in some cases, site-specific conditions may dictate a different approach. It is important to consult with the primacy agency whenever a change in treatment is considered. Remember that the IESWTR requires certain PWSs to use disinfection profiling and benchmarking procedures when proposing changes to disinfection practices. In addition, some of the decisions may lead to a situation where additional treatment will be required. For example, filtration may be needed to reduce the disinfectant dose and

limit DBP formation in cases of high TOC and high bromide ion levels. In those instances, bench scale or pilot studies may be required to select the most appropriate disinfectant.

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# 2. DISINFECTANT USE IN WATER TREATMENT

To comply with the SDWA regulations, the majority of PWSs use some form of water treatment. The 1995 Community Water Systems Survey (USEPA, 1997a) reports that in the United States, 99 percent of surface water systems provide some treatment to their water, with 99 percent of these treatment systems using disinfection/oxidation as part of the treatment process. Although 45 percent of ground water systems provide no treatment, 92 percent of those ground water plants that do provide some form of treatment include disinfection/oxidation as part of the treatment process. The most commonly used disinfectants/oxidants (in no particular order) are chlorine, chlorine dioxide, chloramines, ozone, and potassium permanganate.

Disinfectants are also used to achieve other specific objectives in drinking water treatment. These other objectives include nuisance control (e.g., for zebra mussels and Asiatic clams), oxidation of specific compounds (i.e., taste and odor causing compounds, iron, and manganese), and use as a coagulant and filtration aid.

The purpose of this chapter is to:

- Provide a brief overview of the need for disinfection in water treatment.
- Provide basic information that is common to all disinfectants;
- Discuss other uses for disinfectant chemicals (i.e., as oxidants);
- Describe trends in DBP formation and the health effects of DBPs found in water treatment;
- Discuss microorganisms of concern in water systems, their associated health impact, and the inactivation mechanisms and efficiencies of various disinfectants; and
- Summarize current disinfection practices in the United States, including the use of chlorine as a disinfectant and an oxidant.

## 2.1 Need for Disinfection in Water Treatment

Although the epidemiological relation between water and disease had been suggested as early as the 1850s, it was not until the establishment of the germ theory of disease by Pasteur in the mid-1880s that water as a carrier of disease-producing organisms was understood. In the 1880s, while London experienced the “Broad Street Well” cholera epidemic, Dr. John Snow conducted his now famous epidemiological study. Dr. Snow concluded that the well had become contaminated by a visitor, with the disease, who had arrived in the vicinity. Cholera was one of the first diseases to be

recognized as capable of being waterborne. Also, this incident was probably the first reported disease epidemic attributed to direct recycling of non-disinfected water. Now, over 100 years later, the list of potential waterborne diseases due to pathogens is considerably larger, and includes bacterial, viral, and parasitic microorganisms, as shown in Table 2-1, Table 2-2 and Table 2-3, respectively.

A major cause for the number of disease outbreaks in potable water is contamination of the distribution system from cross connections and back siphonage with non-potable water. However, outbreaks resulting from distribution system contamination are usually quickly contained and result in relatively few illnesses compared to contamination of the source water or a breakdown in the treatment system, which typically produce many cases of illnesses per incident. When considering the number of cases, the major causes of disease outbreaks are source water contamination and treatment deficiencies (White, 1992). For example, in 1993 a Cryptosporidiosis outbreak affected over 400,000 people in Milwaukee, Wisconsin. The outbreak was associated with deterioration in the raw water quality and a simultaneous decrease in the effectiveness of the coagulation-filtration process (Kramer et al., 1996; MacKenzie et al., 1994). Historically, about 46 percent of the outbreaks in the public water systems are found to be related to deficiencies in source water and treatment systems with 92 percent of the causes of illness due to these two particular problems.

All natural waters support biological communities. Because some microorganisms can be responsible for public health problems, biological characteristics of the source water are one of the most important parameters in water treatment. In addition to public health problems, microbiology can also affect the physical and chemical water quality and treatment plant operation.

## 2.1.1 Pathogens of Primary Concern

Table 2-4 shows the attributes of three groups of pathogens of concern in water treatment, namely bacteria, viruses, and protozoa.

### *2.1.1.1 Bacteria*

Bacteria are single-celled organisms typically ranging in size from 0.1 to 10 μm. Shape, components, size, and the manner in which they grow can characterize the physical structure of the bacterial cell. Most bacteria can be grouped by shape into four general categories: spheroid, rod, curved rod or spiral, and filamentous. Cocci, or spherical bacteria, are approximately 1 to 3 μm in diameter. Bacilli (rod-shaped bacteria) are variable in size and range from 0.3 to 1.5 μm in width (or diameter) and from 1.0 to 10.0 μm in length. Vibrios, or curved rod-shaped bacteria, typically vary in size from 0.6 to 1.0 μm in width (or diameter) and from 2 to 6 μm in length. Spirilla (spiral bacteria) can be found in lengths up to 50 μm whereas filamentous bacteria can occur in length in excess of 100 μm.

## Table 2-1. Waterborne Diseases from Bacteria

| Causative Agent | Disease | Symptoms | Reservoir |
|---|---|---|---|
| *Salmonella typhosa* | Typhoid Fever | Headache, nausea, loss of appetite, constipation or diarrhea, insomnia, sore throat, bronchitis, abdominal pain, nose bleeding, shivering and increasing fever. Rose spots on trunk. Incubation period: 7-14 days. | Feces and urine of typhoid carrier or patient. |
| *S. paratyphi*<br>*S. schottinulleri*<br>*S. hirschfeldi* C. | Paratyphoid fever | General infection characterized by continued fever, diarrhea disturbances, sometimes rose spots on trunk. Incubation period: 1-10 days. | Feces and urine of carrier or patient. |
| *Shigella flexneri*<br>*Sh. dysenteriae*<br>*Sh. sonnei*<br>*Sh. paradysinteriae* | Bacillary dysentery | Acute onset with diarrhea, fever, tenesmus and stool frequently containing mucus and blood. Incubation period: 1-7 days. | Bowel discharges of carriers and infected persons. |
| *Vibrio comma*<br>*V. cholerae* | Cholera | Diarrhea, vomiting, rice water stools, thirst, pain, coma. Incubation period: a few hours to 5 days. | Bowel discharges, vomitus, carriers. |
| *Pasteurella tularensis* | Tularemia | Sudden onset with pains and fever; prostration. Incubation period: 1-10 days. | Rodent, rabbit, horsefly, woodtick, dog, fox, hog. |
| Brucella melitensis | Brucellosis (undulant fever) | Irregular fever, sweating, chills, pain in muscles. | Tissues, blood, mold, urine, infected animal. |
| Pseudomonas pseudomallei | Melioidosis | Acute diarrhea, vomiting, high fever, delirium, mania. | Rats, guinea pigs, cats, rabbits, dogs, horses. |
| *Leptospira icterohaemorrhagiae (spirochaetales)* | Leptospirosis (Well's disease) | Fevers, rigors, headaches, nausea, muscular pains, vomiting, thirst, prostration and jaundice may occur. | Urine and feces of rats, swine, dogs, cats, mice, foxes, sheep. |
| Enteropathogenic *E. coli* | Gastroenteritis | Water diarrhea, nausea, prostration and dehydration. | Feces of carrier. |

Sources: Salvato, 1972; Geldreich, 1972.

## Table 2-2. Waterborne Diseases from Human Enteric Viruses

| Group | Subgroup | No. of Types or Subtypes | Disease Entities Associated with These Viruses | Pathological Changes in Patients | Organs Where Virus Multiplies |
|---|---|---|---|---|---|
| Enterovirus | Poliovirus | 3 | Muscular paralysis | Destruction of motor neurons | Intestinal mucosa, spinal cord, brain stem |
| | | | Aseptic meningitis | Inflammation of meninges from virus | Meninges |
| | | | Febrile episode | Viremia and viral multiplication | Intestinal mucosa and lymph |
| | Echovirus | 34 | Aseptic meningitis | Inflammation of meninges from virus | Stem |
| | | | Muscular paralysis | Destruction of motor neurons | Intestinal mucosa, spinal cord, brain |
| | | | Guillain-Barre's Syndrome[1] | Destruction of motor neurons | Spinal cord |
| | | | Exanthem | Dilation and rupture of blood vessels | Skin |
| | | | Respiratory diseases | Viral invasion of parenchymiatous of respiratory tracts and secondary inflammatory responses | Respiratory tracts and lungs |
| | | | | | Gastrointestinal tract |
| | | | Diarrhea | intestinal infections | Respiratory tract and gastrointestinal tract |
| | | | Epidemic myalgia | Not well known | |
| | | | Pericarditis and myocarditis | Viral invasion of cells with secondary inflammatory responses | Pericardial and myocardial tissue |
| | | | Hepatitis | Invasion of parenchyma cells | Liver |
| | Coxsackie-virus | >24 | Herpangina[2] | Viral invasion of mucosa with secondary inflammation | Mouth |
| | A | | Acute lymphatic pharyngitis | Sore throat, pharyngeal lesions | Lymph nodes and pharynx |
| | | | Aseptic meningitis | Inflammation of meninges from virus | Meninges |
| | | | Muscular paralysis | Destruction of motor neurons | Intestinal mucosa, spinal cord, brain stem |
| | | | Hand-foot-mouth disease[3] | Viral invasion of skin cells of hands-feet-mouth | Skin of hands-feet, and much of mouth |
| | | | Respiratory disease | Viral invasion of parenchymiatous of respiratory tracts and secondary inflammatory responses | Respiratory tracts and lungs |
| | | | Infantile diarrhea | Viral invasion of cells of mucosa | Intestinal mucosa |
| | | | Hepatitis | Invasion of parenchyma cells | Liver |
| | | | Pericarditis and myocarditis | Viral invasion of cells with secondary inflammatory responses | Pericardial and myocardial tissue |

[1] Ascending type of muscular paralysis
[2] Febrile episode with sores in mouth
[3] Rash and blister on hand-foot-mouth with fever

## Table 2-2. Waterborne Diseases from Human Enteric Viruses (Continued)

| Group | Subgroup | No. of Types or Subtypes | Disease Entities Associated with These Viruses | Pathological Changes in Patients | Organs Where Virus Multiplies |
|---|---|---|---|---|---|
| Enterovirus (continued) | B | 6 | Pleurodynia[4] | Viral invasion of muscle cells | Intercostal muscles |
| | | | Aseptic meningitis | Inflammation of meninges from virus | Meninges |
| | | | Muscular paralysis | Destruction of motor neurons | Intestinal mucosa, spinal cord, brain stem |
| | | | Meningoencephalitis | Viral invasion of cells | Meninges and brains |
| | | | Pericarditis, endocarditis, myocarditis | Viral invasion of cells with secondary inflammatory responses | Pericardial and myocardial tissue |
| | | | Respiratory disease | Viral invasion of parenchymiatous of respiratory tracts and secondary inflammatory responses | Respiratory tracts and lungs |
| | | | Hepatitis or Rash | Invasion of parenchyma cells | Liver |
| | | | Spontaneous abortion | Viral invasion of vascular cells | Placenta |
| | | | Insulin-dependent diabetes | Viral invasion of insulin-producing cells | Langerhan's cells of pancreases |
| | | | Congenital heart anomalies | Viral invasion of muscle cells | Developing heart |
| Reovirus | | 6 | Not well known | Not well known | |
| Adenovirus | | 31 | Respiratory diseases | Viral invasion of parenchymiatous of respiratory tracts and secondary inflammatory responses | Respiratory tracts and lungs |
| | | | Acute conjunctivitis | Viral invasion of cells and secondary inflammatory responses | Conjunctival cells and blood vessels |
| | | | Acute appendicitis | Viral invasion of mucosa cells | Appendia and lymph nodes |
| | | | Intussusception | Viral invasion of lymph nodes | Intestinal lymph nodes |
| | | | Subacute thyroiditis | Viral invasion of parenchyma cells | Thyroid |
| | | | Sarcoma in hamsters | Sarcoma in hamsters | Muscle cells |
| Hepatitis | | >2 | Infectious hepatitis | Invasion of parenchyma cells | Liver |
| | | | Serum hepatitis | Invasion of parenchyma cells | Liver |
| | | | Down's Syndrome | Invasion of cells | Frontal lobe of brain, muscle, bones |

[4] Pleuritis type of pain with fever

Source: Taylor, 1974; Beneson, 1981.

**Table 2-3. Waterborne Diseases from Parasites**

| Causative Agent | Disease | Symptoms |
|---|---|---|
| *Ascario lumbricoides* (round worm) | Ascariasis | Vomiting, live worms in feces |
| *Cryptosporidium muris* and *parvum* | Cryptosporidiosis | Acute diarrhea, abdominal pain, vomiting, and low-grade fever. Can be life-threatening in immunodeficient patients |
| *Entamoeba histolytica* | Amebiasis | Diarrhea alternating with constipation, chronic dysentery with mucus and blood |
| *Giardia lamblia* | Giardiasis | Intermittent diarrhea |
| *Naegleria gruberi* | Amoebic meningoecephalitis | Death |
| *Schistosoma mansoni* | Schistosomiasis | Liver and bladder infection |
| *Taenia saginata* (beef tapeworm) | Taeniasis | Abdominal pain, digestive disturbances, loss of weight |

Source: Geldreich, 1972; Beneson, 1981.

### 2.1.1.2 Viruses

Viruses are microorganisms composed of the genetic material deoxyribonucleic acid (DNA) or ribonucleic acid (RNA) and a protective protein coat (either single, double, or partially double stranded). All viruses are obligate parasites, unable to carry out any form of metabolism and are completely dependent upon host cells for replication. Viruses are typically 0.01 to 0.1 μm in size and are very species specific with respect to infection, typically attacking only one type of host. Although the principal modes of transmission for the hepatitis B virus and poliovirus are through food, personal contact, or exchange of body fluids, these viruses can be transmitted through potable water. Some viruses, such as the retroviruses (including the HIV group), appear to be too fragile for water transmission to be a significant danger to public health (Riggs, 1989).

### 2.1.1.3 Protozoa

Protozoa are single-cell eucaryotic microorganisms without cell walls that utilize bacteria and other organisms for food. Most protozoa are free-living in nature and can be encountered in water; however, several species are parasitic and live on or in host organisms. Host organisms can vary from primitive organisms such as algae to highly complex organisms such as human beings. Several species of protozoa known to utilize human beings as hosts are shown in Table 2-5.

## Table 2-4. Attributes of the Three Waterborne Pathogens of Concern in Water Treatment

| Organism | Size (μm) | Mobility | Point(s) of Origin | Resistance to Disinfection | Removal by Sedimentation, Coagulation, and Filtration |
|---|---|---|---|---|---|
| Bacteria | 0.1–10 | Motile, Nonmotile | Humans and animals, water, and contaminated food | Type specific - bacterial spores typically have the highest resistance whereas vegetative bacteria have the lowest resistance | Good, 2 to 3-log removal |
| Viruses | 0.01–0.1 | Nonmotile | Humans and animals, polluted water, and contaminated food | Generally more resistant than vegetative bacteria | Poor, 1 to 3-log removal |
| Protozoa | 1–20 | Motile, Nonmotile | Humans and animals, sewage, decaying vegetation, and water | More resistant than viruses or vegetative bacteria | Good, 2 to 3-log removal |

## Table 2-5. Human Parasitic Protozoans

| Protozoan | Host(s) | Disease | Transmission | Occurrence |
|---|---|---|---|---|
| *Acanathamoeba castellannii* | Fresh water, sewage, humans, and soil | Amoebic meningoencephalitis | Gains entry through abrasions, ulcers, and as secondary invader during other infections | North America |
| *Balantidium coli* | Pigs, humans | Balantidiasis (dysentery) | Contaminated water | Micronesia has been the only known site of an outbreak |
| *Cryptosporidium parvum* | Animals, humans | Cryptosporidiosis | Person-to-person or animal-to-person contact, ingestion of fecally contaminated water or food, or contact with fecally contaminated environmental surfaces. | Canada, England, and the United States |
| *Entamoeba histolytica* | Humans | Amoebic dysentery | Contaminated water | Last United States outbreak, 1953 |
| *Giardia lamblia* | Animals, humans | Giardiasis (gastroenteritis) | Contaminated water | Mexico, United States, USSR |
| *Naegleria fowleri* | Soil, water, humans and decaying vegetation | Primary amoebic meningoencephalitis | Nasal inhalation with subsequent penetration of nasopharynx; exposure from swimming in freshwater lakes | North America |

Source: Montgomery, 1985; AWWA, 1995.

## 2.1.2 Recent Waterborne Outbreaks

Within the past 40 years, several pathogenic agents never before associated with documented waterborne outbreaks have appeared in the United States. Enteropathogenic *E. coli* and *Giardia lamblia* were first identified to be the etiological agent responsible for waterborne outbreaks in the 1960s. The first recorded *Cryptosporidium* infection in humans occurred in the mid-1970s. Also during that time was the first recorded outbreak of pneumonia caused by *Legionella pneumophila* (Centers for Disease Control, 1989; Witherell et al., 1988). Recently, there have been numerous documented waterborne disease outbreaks that have been caused by *E. coli, Giardia lamblia, Cryptosporidium,* and *Legionella pneumophila*.

### *2.1.2.1 E. coli*

The first documented case of waterborne disease outbreaks in the United States associated with enteropathogenic *E. coli* occurred in the 1960s. Various serotypes of *E. coli* have been implicated as the etiological agent responsible for disease in newborn infants, usually the result of cross contamination in nurseries. Now, there have been several well-documented outbreaks of *E. coli* (serotypes 0111:B4 and 0124:B27) associated with adult waterborne disease (AWWA, 1990, and Craun, 1981). In 1975, the etiologic agent of a large outbreak at Crater Lake National Park was *E. coli* serotype 06:H16 (Craun, 1981).

### *2.1.2.2 Giardia lamblia*

Similar to *E. coli, Giardia lamblia* was first identified in the 1960s to be associated with waterborne outbreaks in the United States. *Giardia lamblia* is a flagellated protozoan that is responsible for Giardiasis, a disease that can range from being mildly to extremely debilitating. *Giardia* is currently one of the most commonly identified pathogens responsible for waterborne disease outbreaks. The life cycle of *Giardia* includes a cyst stage when the organism remains dormant and is extremely resilient (i.e., the cyst can survive some extreme environmental conditions). Once ingested by a warm-blooded animal, the life cycle of *Giardia* continues with excystation. The cysts are relatively large (8-14 μm) and can be removed effectively by filtration using diatomaceous earth, granular media, or membranes.

Giardiasis can be acquired by ingesting viable cysts from food or water or by direct contact with fecal material. In addition to humans, wild and domestic animals have been implicated as hosts. Between 1972 and 1981, 50 waterborne outbreaks of Giardiasis occurred with about 20,000 reported cases (Craun and Jakubowski, 1986). Currently, no simple and reliable method exists to assay *Giardia* cysts in water samples. Microscopic methods for detection and enumeration are tedious and require examiner skill and patience. *Giardia* cysts are relatively resistant to chlorine, especially at higher pH and low temperatures.

#### 2.1.2.3 Cryptosporidium

*Cryptosporidium* is a protozoan similar to *Giardia*. It forms resilient oocysts as part of its life cycle. The oocysts are smaller than *Giardia* cysts, typically about 4-6 μm in diameter. These oocysts can survive under adverse conditions until ingested by a warm-blooded animal and then continue with excystation.

Due to the increase in the number of outbreaks of Cryptosporidiosis, a tremendous amount of research has focused on *Cryptosporidium* within the last 10 years. Medical interest has increased because of its occurrence as a life-threatening infection to individuals with depressed immune systems. As previously mentioned, in 1993, the largest documented waterborne disease outbreak in United States history occurred in Milwaukee and was determined to be caused by *Cryptosporidium*. An estimated 403,000 people became ill, 4,400 people were hospitalized, and 100 people died. The outbreak was associated with a deterioration in raw water quality and a simultaneous decrease in effectiveness of the coagulation-filtration process, which led to an increase in the turbidity of treated water and inadequate removal of *Cryptosporidium* oocysts.

#### 2.1.2.4 Legionella pneumophila

An outbreak of pneumonia occurred in 1976 at the annual convention of the Pennsylvania American Legion. A total of 221 people were affected by the outbreak, and 35 of those afflicted died. The cause of the pneumonia was not determined immediately despite an intense investigation by the Centers for Disease Control. Six months after the incident, microbiologists were able to isolate a bacterium from the autopsy lung tissue of one of the Legionnaires. The bacterium responsible to the outbreak was found to be distinct from other known bacterium and was named *Legionella pneumophila* (Witherell et al., 1988). Following the discovery of this organism, other *Legionella*-like organisms were discovered. Altogether, 26 species of *Legionella* have been identified, and seven are etiologic agents for Legionnaires' disease (AWWA, 1990).

Legionnaires' disease does not appear to be transferred person-to-person. Epidemiological studies have shown that the disease enters the body through the respiratory system. *Legionella* can be inhaled in water particles less than 5μm in size from facilities such as cooling towers, hospital hot water systems, and recreational whirlpools (Witherell et al., 1988).

### 2.1.3 Mechanism of Pathogen Inactivation

The three primary mechanisms of pathogen inactivation are to:

- Destroy or impair cellular structural organization by attacking major cell constituents, such as destroying the cell wall or impairing the functions of semi-permeable membranes;
- Interfere with energy-yielding metabolism through enzyme substrates in combination with prosthetic groups of enzymes, thus rendering the enzymes non-functional; and

- Interfere with biosynthesis and growth by preventing synthesis of normal proteins, nucleic acids, coenzymes, or the cell wall.

Depending on the disinfectant and microorganism type, combinations of these mechanisms can also be responsible for pathogen inactivation. In water treatment, it is believed that the primary factors controlling disinfection efficiency are: (1) the ability of the disinfectant to oxidize or rupture the cell wall; and (2) the ability of the disinfectant to diffuse into the cell and interfere with cellular activity (Montgomery, 1985).

## 2.2 Other Uses of Disinfectants in Water Treatment

Disinfectants are used for more than just disinfection in drinking water treatment. While inactivation of pathogenic organisms is a primary function, disinfectants are also used oxidants in drinking water treatment for several other functions:

- Minimization of DBP formation;
- Control of nuisance Asiatic clams and zebra mussels;
- Oxidation of iron and manganese;
- Prevention of regrowth in the distribution system and maintenance of biological stability;
- Removal of taste and odors through chemical oxidation;
- Improvement of coagulation and filtration efficiency;
- Prevention of algal growth in sedimentation basins and filters;
- Removal of color.

A brief discussion of these additional oxidant uses follows.

### 2.2.1 Minimization of DBP Formation

Strong oxidants may play a role in disinfection and DBP control strategies in water treatment. Several strong oxidants, including potassium permanganate and ozone, may be used to control DBP precursors.

Potassium permanganate can be used to oxidize organic precursors at the head of the treatment plant, thus minimizing the formation of byproducts at the downstream disinfection stage of the plant. The use of potassium permanganate as an oxidant and disinfectant is discussed in Chapter 5 of this guidance manual.

The use of ozone for oxidation of DBP precursors is currently being studied. Early work has shown that the effects of ozonation, prior to chlorination, were highly site-specific and unpredictable. The key variables that seem to determine the effect of ozone are dose, pH, alkalinity, and the nature of the organic material. Ozone has been shown to be effective for DBP precursor reduction at low pHs. However, at higher pHs (i.e., above 7.5), ozone may actually increase the amount of chlorination byproduct precursors. The use of ozone as an oxidant and disinfectant is addressed in detail in Chapter 3 of this document.

## 2.2.2 Control of Nuisance Asiatic Clams and Zebra Mussels

The Asiatic clam (*Corbicula fluminea)* was introduced to the United States from Southeast Asia in 1938 and now inhabits almost every major river system south of 40° latitude (Britton and Morton, 1982; Counts, 1986). Asiatic clams have been found in the Trinity River, TX; the Ohio River at Evansville, IN; New River at Narrows and Glen Lyn, VA; and the Catawba River in Rock Hill, SC (Belanger et al., 1991; Cameron et al., 1989a; Matisoff et al., 1996). This animal has invaded many water utilities, clogging source water transmission systems and valves, screens, and meters; damaging centrifugal pumps; and causing taste and odor problems (Sinclair, 1964; Evans et al., 1979; Smith, 1979).

Cameron et al. (1989a) investigated the effectiveness of several oxidants to control the Asiatic clam in both the juvenile and adult phases. As expected, the adult clam was found to be much more resistant to oxidants than the juvenile form. In many cases, the traditional method of control, free chlorination, cannot be used because of the formation of excessive amounts of THMs. As shown in Table 2-6, Cameron et al. (1989a) compared the effectiveness of four oxidants for controlling the juvenile Asiatic clam in terms of the LT50 (time required for 50 percent mortality). Monochloramine was found to be the best for controlling the juvenile clams without forming THMs. Further research showed that the effectiveness of monochloramine increased greatly as the temperature increased (Cameron et al., 1989b). Note that the temperatures in this study reflect conditions in the Lynchburg Reservoir, Houston, Texas. Clams can tolerate temperatures between 2 and 35°C (Cameron et al. 1989a).

**Table 2-6. The Effects of Various Oxidants on Mortality of the Asiatic Clam (*Corbicula fluminea*)**

| Chemical | Residual (mg/L) | Temperature (°C) | pH | Life Stage | LT50 (days) |
|---|---|---|---|---|---|
| Free chlorine | 0.5 | 23 | 8.0 | A | 8.7 |
| | 4.8 | 21 | 7.9 | A | 5.9 |
| | 4.7 | 16 | 7.8 | J | 4.8 |
| Potassium Permanganate | 1.1 | 17 | 7.6 | J | 7.9 |
| | 4.8 | 17 | 7.6 | J | 8.6 |
| Monochloramine | 2.6 | 28 | 7.9 | J | 0.6 |
| | 10.7 | 17 | 7.9 | J | 0.5 |
| Chlorine dioxide | 1.2 | 24 | 6.9 | J | 0.7 |
| | 4.7 | 22 | 6.6 | J | 0.6 |

A=Adult; J=Juvenile
Source: Cameron et al., 1989a.

In a similar study, Belanger et al. (1991) studied the biocidal potential of total residual chlorine, monochloramine, monochloramine plus excess ammonia, bromine, and copper for controlling the Asiatic clam. Belanger et al. (1991) showed that monochloramine with excess ammonia was the most effective for controlling the clams at 30°C. Chlorination at 0.25 to 0.40 mg/L total residual chlorine at 20 to 25°C controlled clams of all sizes (LT50 below 28 days) but had minimal effect at 12 to 15°C (as low as zero mortality). As in other studies, the toxicity of all the biocides was highly dependent on temperature and clam size.

The zebra mussel (*Dreissena polymorpha*) is a recent addition to the fauna of the Great Lakes. It was first found in Lake St. Clair in 1988, though it is believed that this native of the Black and Caspian seas, was brought over from Europe in ballast water around 1985 (Herbert et al., 1989). The zebra mussel population in the Great Lakes has expanded very rapidly, both in size and geographical distribution (Roberts, 1990). Lang (1994) reported that zebra mussels have been found in the Ohio River, Cumberland River, Arkansas River, Tennessee River, and the Mississippi River south to New Orleans.

Klerks and Fraleigh (1991) evaluated the effectiveness of hypochlorite, permanganate, and hydrogen peroxide with iron for their effectiveness controlling adult zebra mussels. Both continuous and intermittent 28-day static renewal tests were conducted to determine the impact of intermittent dosing. Intermittent treatment proved to be much less effective than continuous dosing.

The hydrogen peroxide-iron combination (1–5 mg/L with 25 percent iron) was less effective in controlling the zebra mussel than either permanganate or hypochlorite. Permanganate (0.5–2.5 mg $KMnO_4$/L) was usually less effective than hypochlorite (0.5–10 mg $Cl_2$/L).

Van Benschoten et al. (1995) developed a kinetic model to predict the rate of mortality of the zebra mussel in response to chlorine. The model shows the relationship between chlorine residual and temperature on the exposure time required to achieve 50 and 95 percent mortality. Data were

collected for chlorine residuals between 0.5 and 3.0 mg $Cl_2$/L and temperatures from 0.3 to 24°C. The results show a strong dependence on temperature and required contact times ranging from two days to more than a month, depending on environmental factors and mortality required.

Brady et al. (1996) compared the efficiency of chlorine to control growth of zebra mussel and quagga mussel (*Dreissena bugensis*). The quagga mussel is a newly identified mollusk within the Great Lakes that is similar in appearance to the zebra mussel. Full-scale chlorination treatment found a significantly higher mortality for the quagga mussel. The required contact time for 100 percent mortality for quagga and zebra mussels was 23 days and 37 days, respectively, suggesting that chlorination programs designed to control zebra mussels should also be effective for controlling populations of quagga mussels.

Matisoff et al. (1996) evaluated chlorine dioxide ($ClO_2$) to control adult zebra mussels using single, intermittent, and continuous exposures. A single 30-minute exposure to 20 mg/L chlorine dioxide or higher concentration induced at least 50 percent mortality, while sodium hypochlorite produced only 26 percent mortality, and permanganate and hydrogen peroxide were totally ineffective when dosed at 30 mg/L for 30 minutes under the same conditions. These high dosages, even though only used for a short period, may not allow application directly in water for certain applications due to byproducts that remain in the water. Continuous exposure to chlorine dioxide for four days was effective at concentrations above 0.5 mg/L (LC50 = 0.35 mg/L), and 100 percent mortality was achieved at chlorine dioxide concentrations above 1 mg/L.

These experiences all show that the dose required to induce mortality to these nuisance organisms is extremely high, both in terms of chemical dose and contact time. The potential impact on DBPs is significant, especially when the water is high in organic content with a high propensity to form THMs and other DBPs.

## 2.2.3 Oxidation of Iron and Manganese

Iron and manganese occur frequently in ground waters but are less problematic in surface waters. Although not harmful to human health at the low concentrations typically found in water, these compounds can cause staining and taste problems. These compounds are readily treated by oxidation to produce a precipitant that is removed in subsequent sedimentation and filtration processes.

Almost all the common oxidants except chloramines will convert ferrous (2+) iron to the ferric (3+) state and manganese (2+) to the (4+) state, which will precipitate as ferric hydroxide and manganese dioxide, respectively (AWWA, 1990). The precise chemical composition of the precipitate will depend on the nature of the water, temperature, and pH.

Table 2-7 shows that oxidant doses for iron and manganese control are relatively low. In addition, the reactions are relatively rapid, on the order of seconds while DBP formation occurs over hours. Therefore, with proper dosing, residual chlorine during iron and manganese oxidation is therefore relatively low and short lived. These factors reduce the potential for DBP formation as a result of oxidation for iron and manganese removal.

**Table 2-7. Oxidant Doses Required for Oxidation of Iron and Manganese**

| Oxidant | Iron (II) (mg/mg Fe) | Manganese (II) (mg/mg Mn) |
|---|---|---|
| Chlorine, $Cl_2$ | 0.62 | 0.77 |
| Chlorine dioxide, $ClO_2$ | 1.21 | 2.45 |
| Ozone, $O_3$ | 0.43 | 0.88* |
| Oxygen, $O_2$ | 0.14 | 0.29 |
| Potassium permanganate, $KMnO_4$ | 0.94 | 1.92 |

Source: Culp/Wesner/Culp, 1986; Langlais et al., 1991.
* Optimum pH for manganese oxidation using ozone is 8-8.5 Source: Reckhow et al., 1991.

## 2.2.4 Prevention of Regrowth in the Distribution System and Maintenance of Biological Stability

Biodegradable organic compounds and ammonia in treated water can cause microbial growth in the distribution system. "Biological stability" refers to a condition wherein the treated water quality does not enhance biological growth in the distribution system. Biological stability can be accomplished in several ways:

- Removing nutrients from the water prior to distribution;
- Maintaining a disinfectant residual in the treated water; and
- Combining nutrient removal and disinfectant residual maintenance.

To maintain biological stability in the distribution system, the Total Coliform Rule (TCR) requires that treated water have a residual disinfectant of 0.2 mg/L when entering the distribution system. A measurable disinfectant residual must be maintained in the distribution system, or the utility must show through monitoring that the heterotrophic plate count (HPC) remains less than 500/100 mL. A system remains in compliance as long as 95 percent of samples meet these criteria. Chlorine, monochloramine, and chlorine dioxide are typically used to maintain a disinfectant residual in the distribution system. Filtration can also be used to enhance biological stability by reducing the nutrients in the treated water.

The level of secondary disinfectant residual maintained is low, typically in the range of 0.1-0.3 mg/L, depending on the distribution system and water quality. However, because the contact times in the system are quite long, it is possible to generate significant amounts of DBPs in the distribution system, even at low disinfectant doses.

Distribution system problems associated with the use of combined chlorine residual (chloramines), or no residual, have been documented in several instances. The use of combined chlorine is characterized by an initial satisfactory phase in which chloramine residuals are easily maintained throughout the system and bacterial counts are very low. However, problems may develop over a

period of years including increased bacterial counts, reduced combined chlorine residual, increased taste and odor complaints, and reduced transmission main carrying capacity. Conversion of the system to free-chlorine residual produces an initial increase in consumer complaints of taste and odors resulting from oxidation of accumulated organic material. Also, it is difficult to maintain a free-chlorine concentration at the ends of the distribution system (AWWA, 1990).

## 2.2.5 Removal of Taste and Odors Through Chemical Oxidation

Tastes and odors in drinking water are caused by several sources, including microorganisms, decaying vegetation, hydrogen sulfide, and specific compounds of municipal, industrial, or agricultural origin. Disinfectants themselves can also create taste and odor problems. In addition to a specific taste-and odor-causing compound, the sensory impact is often accentuated by a combination of compounds. More recently, significant attention has been given to tastes and odors from specific compounds such as geosmin, 2-methylisoborneol (MIB), and chlorinated inorganic and organic compounds (AWWARF, 1987).

Oxidation is commonly used to remove taste and odor causing compounds. Because many of these compounds are very resistant to oxidation, advanced oxidation processes (ozone/hydrogen peroxide, ozone/UV, etc.) and ozone by itself are often used to address taste and odor problems. The effectiveness of various chemicals to control taste and odors can be site-specific. Suffet et al. (1986) found that ozone is generally the most effective oxidant for use in taste and odor treatment. They found ozone doses of 2.5 to 2.7 mg/L and 10 minutes of contact time (residual 0.2 mg/L) significantly reduce levels of taste and odors. Lalezary et al. (1986) used chlorine, chlorine dioxide, ozone, and permanganate to treat earthy-musty smelling compounds. In that study, chlorine dioxide was found most effective, although none of the oxidants were able to remove geosmin and MIB by more than 40 to 60 percent. Potassium permanganate has been used in doses of 0.25 to 20 mg/L. Studies at the Metropolitan Water District of Southern California demonstrated the effectiveness of peroxone (ozone plus hydrogen peroxide) to remove geosmin and MIB in water treatment (Ferguson et al., 1990; Ferguson et al., 1991; Huck et al., 1995).

Prior experiences with taste and odor treatment indicate that oxidant doses are dependent on the source of the water and causative compounds. In general, small doses can be effective for many taste and odor compounds, but some of the difficult-to-treat compounds require strong oxidants such as ozone and/or advanced oxidation processes or alternative technologies such as granular activated carbon (GAC) adsorption.

## 2.2.6 Improvement of Coagulation and Filtration Efficiency

Oxidants, specifically ozone, have been reported to improve coagulation and filtration efficiency (Gurol and Pidatella, 1983; Farvardin and Collins, 1990; Reckhow et al., 1993; Masschelein, 1992). Others, however, have found no improvement in effluent turbidity from oxidation (Tobiason et al., 1992; Hiltebrand et al., 1986). Prendiville (1986) collected data from a large treatment plant showing that preozonation was more effective than prechlorination to reduce filter effluent turbidities. The cause of

the improved coagulation is not clear, but several possibilities have been offered (Reckhow et al., 1986), including:

- Oxidation of organics into more polar forms;
- Oxidation of metal ions to yield insoluble complexes such as ferric iron complexes; and
- Change in the structure and size of suspended particles.

### 2.2.7 Prevention of Algal Growth in Sedimentation Basins and Filters

Prechlorination is often used to minimize operational problems associated with biological growth in water treatment plants (AWWA, 1990; Culp/Wesner/Culp, 1986). Prechlorination will prevent slime formation on filters, pipes, and tanks, and reduce potential taste and odor problems associated with such slimes. Many sedimentation and filtration facilities operate with a small chlorine residual to prevent growth of algae and bacteria in the launders and on the filter surfaces. This practice has increased in recent years as utilities take advantage of additional contact time in the treatment units to meet disinfection requirements under the SWTR.

### 2.2.8 Removal of Color

Free chlorine is used for color removal. A low pH is favored. Color is caused by humic compounds, which have a high potential for DBP formation. The chlorine dosage and kinetics for color removal are best determined through bench studies.

## 2.3 Disinfection Byproducts and Disinfection Residuals

### 2.3.1 Types of DBPs and Disinfection Residuals

Table 2-8 is a list, compiled by EPA, of DBPs and disinfection residuals that may be of health concern. The table includes both the disinfectant residuals and the specific byproducts produced by the disinfectants of interest in drinking water treatment. These contaminants of concern are grouped into four distinct categories and include disinfectant residuals, inorganic byproducts, organic oxidation byproducts, and halogenated organic byproducts. Tables 1-3 and 1-4 list the disinfection byproducts and disinfectant residuals that are currently regulated.

## Table 2-8. List of Disinfection Byproducts and Disinfection Residuals

| | |
|---|---|
| **DISINFECTANT RESIDUALS** | **HALOGENATED ORGANIC BYPRODUCTS** |
| Free Chlorine | Trihalomethanes |
| Hypochlorous Acid | Chloroform |
| Hypochlorite Ion | Bromodichloromethane |
| Chloramines | Dibromochloromethane |
| Monochloramine | Bromoform |
| Chlorine Dioxide | Haloacetic Acids |
| **INORGANIC BYPRODUCTS** | Monochloroacetic Acid |
| Chlorate Ion | Dichloroacetic Acid |
| Chlorite Ion | Trichloroacetic Acid |
| Bromate Ion | Monobromoacetic Acid |
| Iodate Ion | Dibromoacetic Acid |
| Hydrogen Peroxide | Haloacetonitriles |
| Ammonia | Dichloroacetronitrile |
| **ORGANIC OXIDATION BYPRODUCTS** | Bromochloroacetonitrile |
| Aldehydes | Dibromoacetonitrile |
| Formaldehyde | Trichloroacetonitrile |
| Acetaldehyde | Haloketones |
| Glyoxal | 1,1-Dichloropropanone |
| Hexanal | 1,1,1-Trichloropropanone |
| Heptanal | Chlorophenols |
| Carboxylic Acids | 2-Chlorophenol |
| Hexanoic Acid | 2,4-Dichlorophenol |
| Heptanoic Acid | 2,4,6-Trichlorophenol |
| Oxalic Acid | Chloropicrin |
| Assimilable Organic Carbon | Chloral Hydrate |
| | Cyanogen Chloride |
| | N-Organochloramines |
| | MX* |

* 3-Chloro-4-(dichloromethyl)-5-hydroxy-2(5H)-furanone

The production of DBPs depend on the type of disinfectant, the presence of organic material (e.g., TOC), bromide ion, and other environmental factors as discussed in this manual. By removing DBP precursors, the formation of DBPs can be reduced.

The health effects of DBPs and disinfectants are generally evaluated with epidemiological studies and/or toxicological studies using laboratory animals. Table 2-9 indicates the cancer classifications of both disinfectants and DBPs as of January 1999. The classification scheme used by EPA is shown at the bottom of Table 2-9. The EPA classification scheme for carcinogenicity weighs both animal studies and epidemiologic studies, but places greater weight on evidence of carcinogenicity in humans.

## Table 2-9. Status of Health Information for Disinfectants and DBPs

| Contaminant | Cancer Classification[1] |
|---|---|
| Chloroform | B2 |
| Bromodichloromethane | B2 |
| Dibromochloromethane | C |
| Bromoform | B2 |
| Monochloroacetic Acid | -- |
| Dichloroacetic Acid | B2 |
| Trichloroacetic Acid | C |
| Dichloroacetonitrile | C |
| Bromochloroacetonitrile | -- |
| Dibromoacetonitrile | C |
| Trichloroacetonitrile | -- |
| 1,1-Dichloropropanone | -- |
| 1,1,1-Trichloropropanone | -- |
| 2-Chlorophenol | D |
| 2,4-Dichlorophenol | D |
| 2,4,6-Trichlorophenol | B2 |
| Chloropicrin | -- |
| Chloral Hydrate | C |
| Cyanogen Chloride | -- |
| Formaldehyde | B1[2] |
| Chlorate | -- |
| Chlorite | D |
| Bromate | B2 |
| Ammonia | D |
| Hypochlorous Acid | -- |
| Hypochlorite | -- |
| Monochloramine | -- |
| Chlorine Dioxide | D |

(1) The scheme for categorizing chemicals according to their carcinogenic potential is as follows:*

| | |
|---|---|
| Group A: Human Carcinogen | Sufficient evidence in epidemiologic studies to support casual association between exposure and cancer. |
| Group B: Probable Human Carcinogen | Limited evidence in epidemiologic studies (Group B1) and/or sufficient evidence from animal studies (Group B2) |
| Group C: Possible Human Carcinogen | Limited evidence from animal studies and inadequate or no data in humans |
| Group D: Not Classifiable | Inadequate or no human and animal evidence of carcinogenicity |
| Group E: No Evidence of Carcinogenicity for Humans | No evidence of carcinogenicity in at least two adequate animal tests in different species or in adequate epidemiologic and animal studies. |

* EPA is in the process of revising the Cancer Guidelines
Source: USEPA, 1996

(2) Based on inhalation exposure.

## 2.3.2 Disinfection Byproduct Formation

Halogenated organic byproducts are formed when natural organic matter (NOM) reacts with free chlorine or free bromine. Free chlorine can be introduced to water directly as a primary or secondary disinfectant, with chlorine dioxide, or with chloramines. Free bromine results from the oxidation of the bromide ion in source water. Factors affecting formation of halogenated DBPs include type and concentration of natural organic matter, oxidant type and dose, time, bromide ion concentration, pH, organic nitrogen concentration, and temperature. Organic nitrogen significantly influences the formation of nitrogen containing DBPs such as the haloacetonitriles, halopicrins, and cyanogen halides (Reckhow et al., 1990; Hoigné and Bader, 1988). The parameter TOX represents the concentration of total organic halides in a water sample (calculated as chloride). In general, less than 50 percent of the TOX content has been identified, despite evidence that several of these unknown halogenated byproducts of water chlorination may be harmful to humans (Singer and Chang, 1989).

Non-halogenated DBPs are also formed when strong oxidants react with organic compounds found in water. Ozone and peroxone oxidation of organics leads to the production of aldehydes, aldo- and keto-acids, organic acids, and, when bromide ion is present, brominated organics (Singer, 1992). Many of the oxidation byproducts are biodegradable and appear as biodegradable dissolved organic carbon (BDOC) and assimilable organic carbon (AOC) in treated water.

Bromide ion plays a key role in DBP formation. Ozone or free chlorine oxidizes bromide ion to hypobromate ion/hypobromous acid, which subsequently forms brominated DBPs. Brominated organic byproducts include compounds such as bromoform, brominated acetic acids and acetonitriles, bromopicrin, and cyanogen bromide. Only about one third of the bromide ions incorporated into byproducts has been identified.

### *2.3.2.1 Disinfection Byproduct Precursors*

Numerous researchers have documented that NOM is the principal precursor of organic DBP formation (Stevens et al., 1976; Babcock and Singer 1979; Christman et al., 1983). Chlorine reacts with NOM to produce a variety of DBPs, including THMs, haloacetic acids (HAAs), and others. Ozone reacts with NOM to produce aldehydes, organic acids, and aldo- and keto-acids; many of these are produced by chlorine as well (Singer and Harrington, 1993).

Natural waters contain mixtures of both humic and nonhumic organic substances. NOM can be subdivided into a hydrophobic fraction composed of primarily humic material, and a hydrophilic fraction composed of primarily fulvic material.

The type and concentration of NOM are often assessed using surrogate measures. Although surrogate parameters have limitations, they are used because they may be measured more easily, rapidly, and inexpensively than the parameter of interest, often allowing on-line monitoring of the operation and performance of water treatment plants. Surrogates used to assess NOM include:

- Total and dissolved organic carbon (TOC and DOC);

- Specific ultraviolet light absorbance (SUVA), which is the absorbance at 254 nm wavelength (UV-254) divided by DOC (SUVA = (UV-254/DOC)*100, in L/mg-m);
- THM formation potential (THMFP) -- a test measuring the quantity of THMs formed with a high dosage of free chlorine and a long reaction time; and
- TTHM Simulated Distribution System (SDS) -- a test to predict the TTHM concentration at some selected point in a given distribution system, where the conditions of the chlorination test simulate the distribution system at the point desired.

On average, about 90 percent of the TOC is dissolved. DOC is defined as the TOC able to pass through a 0.45 μm filter. UV absorbance is a good technique for assessing the presence of DOC because DOC primarily consists of humic substances, which contain aromatic structures that absorb light in the UV spectrum. Oxidation of DOC reduces the UV absorbance of the water due to oxidation of some of the organic bonds that absorb UV absorbance. Complete mineralization of organic compounds to carbon dioxide usually does not occur under water treatment conditions; therefore, the overall TOC concentration usually is constant.

DBP concentrations vary seasonally and are typically greatest in the summer and early fall for several reasons:

- The rate of DBP formation increases with increasing temperature (Singer et al., 1992);
- The nature of organic DBP precursors varies with season (Singer et al., 1992); and
- Due to warmer temperatures, chlorine demand may be greater during summer months requiring higher dosages to maintain disinfection.

If the bromide ion is present in source waters, it can be oxidized to hypobromous acid that can react with NOM to form brominated DBPs, such as bromoform. Furthermore, under certain conditions, ozone may react with the hypobromite ion ($OBr^-$) to form bromate ion ($BrO^-_3$).

The ratio of bromide ion to the chlorine dose affects THM formation and bromine substitution of chlorine. Increasing the bromide ion to chlorine dose ratio shifts the speciation of THMs to produce more brominated forms (Krasner et al., 1989; Black et al., 1996). In the Krasner et al. study, the chlorine dose was roughly proportional to TOC concentration. As TOC was removed through the treatment train, the chlorine dose decreased and TTHM formation declined. However, at the same time, the bromide ion to chlorine dose increased, thereby shifting TTHM concentrations to the more brominated THMs. Therefore, improving the removal of NOM prior to chlorination can shift the speciation of halogenated byproducts toward more brominated forms.

Chloropicrin is produced by the chlorination of humic materials in the presence of nitrate ion (Duguet et al., 1985; Thibaud et al., 1987). Thibaud et al. (1988) chlorinated humic compounds in the presence of bromide ion to demonstrate the formation of brominated analogs to chloropicrin.

### 2.3.2.2 Impacts of pH on DBP Formation

The pH of water being chlorinated has an impact on the formation of halogenated byproducts as shown in Table 2-10 (Reckhow and Singer, 1985; Stevens et al., 1989). THM formation increases with increasing pH. Trichloroacetic acid, dichloroacetonitrile, and trichloropropanone formation decrease with increased pH. Overall TOX formation decreases with increasing pH.

Based on chlorination studies of humic material in model systems, high pH tends to favor chloroform formation over the formation of trichloroacetic acid and other organic halides. Accordingly, water treatment plants practicing precipitative softening at pH values greater than 9.5 to 10 are likely to have a higher fraction of TOX attributable to THMs than plants treating surface waters by conventional treatment in pH ranges of 6 to 8 (Singer and Chang, 1989).

Since the application of chlorine dioxide and chloramines may introduce free chlorine into water, chlorination byproducts that may be formed would be influenced by pH as discussed above. Ozone application to bromide ion containing waters at high pH favors the formation of bromate ion, while application at low pH favors the formation of brominated organic byproducts. See discussion under individual disinfectants for a more detailed discussion on pH impacts on DBP formation.

The pH also impacts enhanced coagulation (i.e., for ESWTR compliance) and Lead and Copper Rule Compliance. These issues are addressed in EPA's *Microbial and Disinfection Byproduct Simultaneous Compliance Guidance Manual* (expected to be available in 1999).

### 2.3.2.3 Organic Oxidation Byproducts

Organic oxidation byproducts are formed by reactions between NOM and all oxidizing agents added during drinking water treatment. Some of these byproducts are halogenated, as discussed in the previous section, while others are not. The types and concentrations of organic oxidation byproducts produced depend on the type and dosage of the oxidant being used, chemical characteristics and concentration of the NOM being oxidized, and other factors such as the pH and temperature.

Specific chemical byproducts belonging to the classification of halogenated organic oxidation products are listed in Table 2-10. As presented in Table 2-10, the formation of DBPs is pH dependent. Comparisons in the table are made to the formation of TTHMs at a pH of 7.0. AOC is not a specific organic contaminant, but a generally used surrogate measure of bacterial regrowth potential in distribution systems. AOC is comprised of many chemical species, including the aldehydes and carboxylic acids listed in Table 2-8. AOC formation studies, primarily performed in the Netherlands, have shown that both ozonation and chlorination can increase concentrations of AOC. This increase in AOC concentration is believed to be the result of oxidizing high molecular weight organics to smaller and more readily bioassimilable molecules. Because AOC is not a specific chemical contaminant, no specific health effects are attributable to AOC.

**Table 2-10. Conditions of Formation of DBPs**

| By-product | Conditions of Formation | | |
|---|---|---|---|
| | Chlorination at pH 5.0 | Chlorination at pH 7.0 | Chlorination at pH 9.4 |
| Total Trihalomethanes | Lower formation | | Higher formation |
| Trichloroacetic Acid | Similar formation to that at pH 7.0 | Similar formation to that at pH 5.0 | Lower formation |
| Dichloroacetic Acid | Similar formation to that at pH 5.0 and 9.4 - perhaps slightly higher at pH 7.0 | Similar formation to that at pH 5.0 and 9.4- perhaps slightly higher at pH 7.0 | Similar formation to that at pH 5.0 and 7.0 - perhaps slightly higher at pH 7.0 |
| Monochloroacetic Acid | At concentrations <5 μg/L, trends not discernible | At concentrations <5 μg/L, trends not discernible | At concentrations <5 μg/L, trends not discernible |
| Dibromoacetic Acid | At concentrations <1 μg/L, trends not discernible | At concentrations <1 μg/L, trends not discernible | At concentrations <1 μg/L, trends not discernible |
| Chloral Hydrate | Similar formation to that at pH 7.0 | Similar formation to that at pH 5.0 | Forms within 4 hours; decays over time to <5 μg/L |
| Chloropicrin | At concentrations <1 μg/L, trends not discernible | At concentrations <1 μg/L, trends not discernible | At concentrations <1 μg/L, trends not discernible |
| Dichloroacetonitrile | Higher formation | Forms within 4 hours; decays over time to <5 μg/L | Concentrations <2 μg/L, trends not discernible |
| Bromochloroacetonitrile | At concentrations <2 μg/L, trends not discernible | At concentrations <2 μg/L, trends not discernible | At concentrations <2 μg/L, trends not discernible |
| Dibromoacetonitrile | At concentrations <.5 μg/L, trends not discernible | At concentrations <.5 μg/L, trends not discernible | At concentrations <.5 μg/L, trends not discernible |
| Trichloroacetonitrile | Not detected | Not detected | Not detected |
| 1,1,1-Trichloropropanone | Higher formation | At concentrations <2 μg/L, trends not discernible | Not detected |

Source: Stevens et al., 1989.

### *2.3.2.4 Inorganic Byproducts and Disinfectants*

Table 2-11 shows some of the inorganic DBPs that are produced or remain as residual during disinfection. As discussed earlier, bromide ion reacts with strong oxidants to form bromate ion and other organic DBPs. Chlorine dioxide and chloramines leave residuals that are of concern for health considerations, as well as for taste and odor. The significance of these compounds is discussed further in subsequent chapters.

**Table 2-11. Inorganic DBPs Produced During Disinfection**

| Disinfectant | Inorganic Byproduct or Disinfectant Residual Discussed |
|---|---|
| Chlorine Dioxide | Chlorine Dioxide, Chlorite ion, Chlorate ion, Bromate ion (in presence of light) |
| Ozone | Bromate ion, Hydrogen Peroxide |
| Chloramination | Monochloramine, Dichloramine, Trichloramine, Ammonia, Cyanogen Chloride |

## 2.3.3 DBP Control Strategies

In 1983, the EPA identified technologies, treatment techniques, and plant modifications that community water systems could use to comply with the maximum contaminant level for TTHMs. The principal treatment modifications involved moving the point of chlorination downstream in the water treatment plant, improving the coagulation process to enhance the removal of DBP precursors, and using chloramines to supplement or replace the use of free chlorine (Singer, 1993). Moving the point of chlorination downstream in the treatment train often is very effective in reducing DBP formation, because it allows the NOM precursor concentration to be reduced during treatment prior to chlorine addition. Replacing prechlorination by preoxidation with an alternate disinfectant that produces less DBPs is another option for reducing formation of chlorinated byproducts.

Other options to control the formation of DBPs include; source water quality control, DBP precursor removal, and disinfection strategy selection. An overview of each is provided below.

### *2.3.3.1 Source Water Quality Control*

Source water control strategies involve managing the source water to lower the concentrations of NOM and bromide ion in the source water. Research has shown that algal growth leads to the production of DBP precursors (Oliver and Shindler, 1980; Wachter and Andelman, 1984; Karimi and Singer, 1991). Therefore, nutrient and algal management is one method of controlling DBP formation potential of source waters. Control of bromide ion in source waters may be accomplished by preventing brine or salt water intrusion into the water source.

### *2.3.3.2 DBP Precursor Removal*

Raw water can include DBP precursors in both dissolved and particulate forms. For the dissolved precursors to be removed in conventional treatment, they must be converted to particulate form for subsequent removal during settling and filtering. The THM formation potential generally decreases by about 50 percent through conventional coagulation and settling, indicating the importance of moving the point of chlorine application after coagulation and settling (and even filtration) to control TOX as well as TTHM formation (Singer and Chang, 1989). Conventional systems can lower the DBP formation potential of water prior to disinfection by further removing precursors with enhanced coagulation, GAC adsorption, or membrane filtration prior to disinfection. Precursor removal efficiencies are site-specific and vary with different source waters and treatment techniques.

Aluminum (alum) and iron (ferric) salts can remove variable amounts of NOM. For alum, the optimal pH for NOM removal is in the range of 5.5 to 6.0. The addition of alum decreases pH and may allow the optimal pH range to be reached without acid addition. However, waters with very low or very high alkalinities may require the addition of base or acid to reach the optimal NOM coagulation pH (Singer, 1992).

GAC adsorption can be used following filtration to remove additional NOM. For most applications, empty bed contact times in excess of 20 minutes are required, with regeneration frequencies on the order of 2 to 3 months (Singer, 1992). These long contact times and frequent regeneration requirements make GAC an expensive treatment option. In cases where prechlorination is practiced, the chlorine rapidly degrades GAC. Addition of a disinfectant to the GAC bed can result in specific reactions in which previously absorbed compounds leach into the treated water.

Membrane filtration has been shown effective in removing DBP precursors in some instances. In pilot studies, ultrafiltration (UF) with a molecular weight cutoff (MWCO) of 100,000 daltons was ineffective for controlling DBP formation. However, when little or no bromide ion was present in source water, nanofiltration (NF) membranes with MWCOs of 400 to 800 daltons effectively controlled DBP formation (Lâiné et al., 1993). In waters containing bromide ion, higher bromoform concentrations were observed after chlorination of membrane permeate (compared with raw water). This occurs as a result of filtration removing NOM while concentrating bromide ions in the permeate thus providing a higher ratio of bromide ions to NOM than in raw water. This reduction in chlorine demand increases the ratio of bromide to chlorine, resulting in higher bromoform concentrations after chlorination of NF membrane permeate (compared with the raw water). TTHMs were lower in chlorinated permeate than chlorinated raw water. However, due to the shift in speciation of THMs toward more brominated forms, bromoform concentrations were actually greater in chlorinated treated water than in chlorinated raw water. Use of spiral-wound NF membranes (200–300 daltons) more effectively controlled the formation of brominated THMs, but pretreatment of the water was necessary (Lâiné et al., 1993). Significant limitations in the use of membranes are disposal of the waste brine generated, fouling of membranes, cost of membrane replacement, and increasing energy cost.

The promulgated DBPR requires enhanced coagulation as an initial step for removal of DBP precursors. In addition to meeting MCLs and MRDLs, some water suppliers also must meet treatment requirements to control the organic material (DBP precursors) in the raw water that combines with disinfectant residuals to form DBPs. Systems using conventional treatment are required to control precursors (measured as TOC) by using enhanced coagulation or enhanced softening. A system must remove a specified percentage of TOC (based on raw water quality) prior to the point of continuous disinfection (Table 2-12).

Systems using ozone followed by biologically active filtration or chlorine dioxide that meet specific criteria would be required to meet the TOC removal requirements prior to addition of a residual disinfectant. Systems able to reduce TOC by a specified percentage level have met the DBPR treatment technique requirement.

**Table 2-12. Required Removal of TOC by Enhanced Coagulation for Surface Water Systems[+] Using Conventional Treatment[++] (percent reduction)**

| Source Water TOC (mg/L) | Source Water Alkalinity (mg/L as $CaCO_3$) | | |
|---|---|---|---|
| | 0-60 | >60-120 | >120[+++] |
| >2.0-4.0 | 35.0 | 25.0 | 15.0 |
| >4.0-8.0 | 45.0 | 35.0 | 25.0 |
| >8.0 | 50.0 | 40.0 | 30.0 |

\+ Also applies to utilities that treat ground water under the influence of surface water.

++ Systems meeting at least one of the conditions in 40 CFR §§ 141.135(a)(1)(I)-(iv) are not required to operate with enhanced coagulation.

+++ Systems practicing precipitative softening must meet the TOC removal requirements in this column.

If the system does not meet the percent reduction, it must determine its alternative minimum TOC removal level. The primacy agency approves the alternative minimum TOC removal possible for the system on the basis of the relationship between coagulant dose and TOC in the system based on results of bench or pilot-scale testing. Enhanced coagulation is determined in part as the coagulant dose where an incremental addition of 10 mg/L of alum (or an equivalent amount of ferric salt) results in a TOC removal below 0.3 mg/L.

#### *2.3.3.3 Disinfection Strategy Selection*

In addition to improving the raw or predisinfectant water quality, alternative disinfection strategies can be used to control DBPs. These strategies include the following:

- Use an alternative or supplemental disinfectant or oxidant such as chloramines or chlorine dioxide that will produce fewer DBPs;
- Move the point of chlorination to reduce TTHM formation and, where necessary, substitute chloramines, chlorine dioxide, or potassium permanganate for chlorine as a preoxidant;
- Use two different disinfectants or oxidants at various points in the treatment plant to avoid DBP formation at locations where precursors are still present in high quantities;
- Use of powdered activated carbon for THM precursor or TTHM reduction seasonally or intermittently; and
- Maximize precursor removal.

## 2.3.4 CT Factor

One of the most important factors for determining or predicting the germicidal efficiency of any disinfectant is the CT factor, a version of the Chick-Watson law (Chick, 1908; Watson, 1908). The CT factor is defined as the product of the residual disinfectant concentration, C, in mg/L, and the contact time, T, in minutes, that residual disinfectant is in contact with the water.

EPA developed CT values for the inactivation of *Giardia* and viruses under the SWTR. Table 2-13 compares the CT values for virus inactivation using chlorine, chlorine dioxide, ozone, chloramine, and ultraviolet light disinfection under specified conditions. Table 2-14 shows the CT values for inactivation of *Giardia* cyst using chlorine, chloramine, chlorine dioxide, and ozone under specified conditions. The CT values shown in Table 2-13 and Table 2-14 are based on water temperatures of 10°C and pH values in the range of 6 to 9. CT values for chlorine disinfection are based on a free chlorine residual. Note that chlorine is less effective as pH increases from 6 to 9. In addition, for a given CT value, a low C and a high T is more effective than the reverse (i.e., a high C and a low T). For all disinfectants, as temperature increases, effectiveness increases.

**Table 2-13. CT Values for Inactivation of Viruses**

| Disinfectant | Units | Inactivation | | |
|---|---|---|---|---|
| | | 2-log | 3-log | 4-log |
| Chlorine[1] | mg · min/L | 3 | 4 | 6 |
| Chloramine[2] | mg · min/L | 643 | 1,067 | 1,491 |
| Chlorine Dioxide[3] | mg · min/L | 4.2 | 12.8 | 25.1 |
| Ozone | mg · min/L | 0.5 | 0.8 | 1.0 |
| UV | $mW \cdot s/cm^2$ | 21 | 36 | not available |

CT values were obtained from AWWA, 1991.
[1] Values are based on a temperature of 10°C, pH range of 6 to 9, and a free chlorine residual of 0.2 to 0.5 mg/L.
[2] Values are based on a temperature of 10°C and a pH of 8.
[3] Values are based on a temperature of 10°C and a pH range of 6 to 9.

**Table 2-14. CT Values for Inactivation of *Giardia* Cysts**

| Disinfectant | Inactivation (mg · min/L) | | | | | |
|---|---|---|---|---|---|---|
| | 0.5-log | 1-log | 1.5-log | 2-log | 2.5-log | 3-log |
| Chlorine[1] | 17 | 35 | 52 | 69 | 87 | 104 |
| Chloramine[2] | 310 | 615 | 930 | 1,230 | 1,540 | 1,850 |
| Chlorine Dioxide[3] | 4 | 7.7 | 12 | 15 | 19 | 23 |
| Ozone[3] | 0.23 | 0.48 | 0.72 | 0.95 | 1.2 | 1.43 |

CT values were obtained from AWWA, 1991.
[1] Values are based on a free chlorine residual less than or equal to 0.4 mg/L, temperature of 10°C, and a pH of 7.
[2] Values are based on a temperature of 10°C and a pH in the range of 6 to 9.
[3] Values are based on a temperature of 10°C and a pH of 6 to 9.

## 2.4 Pathogen Inactivation Versus DBP Formation

Table 2-15 presents a summary of disinfection parameter impacts on pathogen inactivation and DBP formation.

**Table 2-15. Summary of Disinfection Impacts**

| Disinfection Parameter | Typical Impact on Pathogen Inactivation | Typical Impact on DBP Formation |
|---|---|---|
| Disinfectant Type | Depends on inactivation efficacy | Depends on disinfectant reactivity |
| Disinfectant Strength | The stronger the disinfectant, the quicker the disinfection process. | The stronger the disinfectant, the greater the amount of DBPs. |
| Disinfectant Dose | Increasing the disinfectant dose increases the disinfection rate. | Increasing the disinfectant dose typically increases the rate of DBP formation. |
| Type of Organism | Susceptibility to disinfection varies according to pathogen group. In general, protozoa are more resistant to disinfectants than bacteria and viruses. | None. |
| Contact Time | Increasing the contact time decreases the disinfectant dose required for a given level of inactivation. | Increasing contact time with an equivalent disinfectant dose increases the formation of DBPs. |
| pH | pH may affect the disinfectant form and, in-turn, the efficiency of the disinfectant. | The impact of pH varies with DBP. See Section 2.3.2.3 for a brief summary of relationships between pH and DBP formation. |
| Temperature | Increasing the temperature increases the rate of disinfection. | Increasing temperature is typically associated with faster oxidation kinetics, hence, increased DBP formation. |
| Turbidity | Particles responsible for turbidity can surround and shield pathogenic microorganisms from disinfectants. | Increased turbidity may be associated with increased NOM, which represents an increased amount of DBP precursors for the formation of DBPs when disinfectant is applied. |
| Dissolved Organics | Dissolved organics can interfere with disinfection by creating a demand and reducing the amount of disinfectant available for pathogen inactivation. | Increased dissolved organics will represent a larger amount of DBP precursor for the formation of DBPs when disinfectant is applied. |

## 2.5 Disinfectant Residual Regulatory Requirements

One of the most important factors for evaluating the merits of alternative disinfectants is their ability to maintain the microbial quality in the water distribution system. Disinfectant residuals may serve to protect the distribution system against regrowth (Snead et al., 1980). The SWTR requires that filtration and disinfection must be provided to ensure that the total treatment of the system achieves at least a 3-log removal/inactivation of *Giardia* cysts and a 4-log removal/inactivation of viruses. In addition, the disinfection process must demonstrate by continuous monitoring and recording that the disinfectant residual in the water entering the distribution system is never less than 0.2 mg/L for more than 4 hours.

Several of the alternative disinfectants examined in this manual cannot be used to meet the residual requirements stated in the SWTR. For example, if either ozone or ultraviolet light disinfection are used as the primary disinfectant, a secondary disinfectant such as chlorine or chloramines should be utilized to obtain a residual in the distribution system.

DBP formation continues in the distribution system due to reactions between the residual disinfectant and organics in the water. Koch et al. (1991) found that with a chlorine dose of 3-4 mg/L, THM and HAA concentrations increase rapidly during the first 24 hours in the distribution system. After the initial 48 hours, the subsequent increase in THMs is very small. Chloral hydrate concentrations continued to increase after the initial 24 hours, but at a reduced rate. Haloketones actually decreased in the distribution system.

Nieminski et al. (1993) evaluated DBP formation in the simulated distribution systems of treatment plants in Utah. Finished water chlorine residuals ranged from 0.4 to 2.8 mg/L. Generally, THM values in the distribution system studies increased by 50 to 100 percent (range of 30 to 200 percent) of the plant effluent value after 24-hour contact time. The 24-hour THM concentration was essentially the same as the 7-day THM formation potential. HAA concentrations in the simulated distribution system was about 100 percent (range of 30 to 200 percent) of the HAA in the plant effluent. The 7-day HAA formation potential was sometimes higher, or below the distribution system values. If chlorine is used as a secondary disinfectant, one should therefore anticipate a 100-percent increase in the plant effluent THMs, or plan to reach the 7-day THM formation level in the distribution system.

## 2.6 Summary of Current National Disinfection Practices

Most water treatment plants disinfect water prior to distribution. The 1995 Community Water Systems Survey (USEPA, 1997a) reports that 81 percent of all community water systems provide some form of treatment on all or a portion of their water sources. The survey also found that virtually all surface water systems provide some treatment of their water. Of those systems reporting no treatment, 80 percent rely on ground water as their only water source.

The most commonly used disinfectants/oxidants are chlorine, chlorine dioxide, chloramines, ozone, and potassium permanganate. Table 2-16 shows a breakdown on the chemical usage from the survey. Note that the table shows the percentages of systems using the particular chemical as either disinfectant or some other role. The table shows the predominance of chlorine in surface and ground water disinfection treatment systems with more than 60 percent of the treatment systems using chlorine as disinfectant/oxidant. Potassium permanganate on the other hand, is used by many systems, but its application is primarily for oxidation, rather than for disinfection.

Permanganate will have some beneficial impact on disinfection since it is a strong oxidant that will reduce the chemical demand for the ultimate disinfection chemical. Chloramine is used by some systems and is more frequently used as a post-treatment disinfectant.

The International Ozone Association conducted a survey of ozone facilities in the United States (IOA, 1997). The survey documented the types of ozone facilities, size, objective of ozone application, and year of operation. Table 2-17 summarizes the findings from the survey. The most common use for ozone is for oxidation of iron and manganese, and for taste and odor control. Twenty-four of the 158 ozone facilities used GAC following ozonation. In addition to the 158

operating ozone facilities, the survey identified 19 facilities under construction and another 30 under design. The capacity of the systems range from less than 25 gpm to exceeding 500 mgd. Nearly half of the operating facilities have a capacity exceeding 1 mgd. Rice et al. (1998) found that as of May 1998, 264 drinking water plants in the United States are using ozone.

## Table 2-16. Disinfection Practices of Water Systems that Include Some Form of Treatment

| Treatment | Service Population | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | <100 | 101-500 | 501-1,000 | 1,001-3,300 | 3,301-10,000 | 10,001-50,000 | 50,001-100,000 | Over 100,001 | Total |
| **Surface Water Systems** | | | | | | | | | |
| **Total Number of Systems** | 218 | 432 | 330 | 845 | 679 | 626 | 103 | 104 | 3,337 |
| **Pre-Disinfection, Oxidation/Softening** | | | | | | | | | |
| Chlorine | 59.0% | 73.9% | 67.3% | 66.3% | 68.8% | 58.6% | 47.5% | 57.1% | 63.8% |
| Chlorine dioxide | 0 | 0 | 0 | 5.0 | 4.7 | 13.2 | 14.2 | 7.8 | 6.3 |
| Chloramines | 4.6 | 0 | 1.1 | 2.1 | 0 | 2.2 | 15.5 | 10.8 | 3.1 |
| Ozone | 0 | 0 | 0 | 0 | 0.3 | 0 | 5.4 | 5.8 | 0.9 |
| $KMnO_4$ | 0 | 4.9 | 9.6 | 9.9 | 15.2 | 28.3 | 25.9 | 28.5 | 16.0 |
| Predisinfection/oxidation | 0 | 0 | 2.0 | 2.9 | 0.6 | 9.2 | 5.1 | 4.3 | 3.5 |
| Lime/Soda ash softening | 6.8 | 9.8 | 20.9 | 16.2 | 14.3 | 11.7 | 3.5 | 5.9 | 12.5 |
| Recarbonation | 0 | 0 | 0 | 0 | 2.1 | 4.7 | 0.6 | 6.3 | 1.9 |
| **Post-Disinfection** | | | | | | | | | |
| Chlorine | 49.7 % | 51.6% | 80.6% | 62.8% | 77.9% | 71.1% | 73.8 % | 63.6 % | 67.5 % |
| Chlorine dioxide | 0 | 0 | 0 | 0 | 0.3 | 4.9 | 5.9 | 11.2 | 1.6 |
| Chloramines | 0 | 0 | 0 | 2.9 | 2.1 | 15.6 | 29.4 | 24.2 | 8.1 |
| Postdisinfection combinations | 0 | 0 | 0 | 2.1 | 4.0 | 3.9 | 1.9 | 1.6 | 3.0 |
| **Ground Water Systems** | | | | | | | | | |
| **Total Number of Systems** | 9,042 | 10,367 | 4,443 | 4,422 | 2,035 | 1,094 | 120 | 56 | 31,579 |
| **Pre-Disinfection, Oxidation/Softening** | | | | | | | | | |
| Chlorine | 64.2 % | 69.9 % | 56.7 % | 73.2 % | 60.6 % | 57.4 % | 36.2 % | 38.1 % | 63.9 % |
| Chlorine dioxide | 1.3 | 0 | 0 | 0 | 0 | 0 | 3.1 | 0 | 0.3 |
| Chloramines | 0 | 0 | 0 | 0 | 0 | 0.6 | 1.4 | 0.7 | 0.1 |
| Ozone | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0.6 | 0 |
| $KMnO_4$ | 0 | 0.9 | 2.2 | 0.6 | 5.8 | 3.2 | 7.0 | 0 | 1.8 |
| Predisinfection/oxidation | 0.3 | 0.5 | 0 | 0.7 | 1.0 | 2.6 | 0 | 0 | 0.7 |
| Lime/Soda ash softening | 2.9 | 2.9 | 2.2 | 3.6 | 3.5 | 3.8 | 5.0 | 9.1 | 3.2 |
| Recarbonation | 0 | 0.5 | 0 | 0.6 | 1.4 | 1.5 | 2.8 | 1.1 | 0.6 |
| **Post-Disinfection** | | | | | | | | | |
| Chlorine | 23.0 % | 23.4% | 32.5% | 28.3 % | 42.5% | 41.9 % | 54.5 % | 65.8 % | 31.0 % |
| Chlorine dioxide | 0 | 1.0 | 0 | 0 | 0 | 0.6 | 0 | 0 | 0.4 |
| Chloramines | 0 | 0 | 0 | 0 | 0.1 | 1.1 | 3.9 | 4.3 | 0.3 |
| Postdisinfection combinations | 0 | 0 | 0 | 0 | 0.1 | 0.1 | 0 | 0 | 0 |

Source: USEPA, 1997a.

**Table 2-17. Ozone Application in Water Treatment Plants in the United States**

| Ozone Objective | Number of Plants | % Plants |
|---|---|---|
| THM Control | 50 | 32 |
| Disinfection | 63 | 40 |
| Iron/Manganese, Taste and Odor Control | 92 | 58 |
| Total | 158 | -- |

Source: IOA, 1997.

# 2.7 Chlorine

Although chlorine is not a focus of this guidance manual, the following section provides a brief overview of chlorine use in the water treatment industry to compare with the alternative disinfectants discussed in this manual. Since there is a wealth of excellent literature on chlorine's uses and performance capabilities, summarizing this large body of knowledge here is neither practical nor necessary (see, for example: White, 1992; Chlorine Institute, 1996; and Connell, 1996).

One of the recent developments in chlorine disinfection is the use of multiple and interactive disinfectants. In these applications, chlorine is combined with a second disinfectant to achieve improved disinfection efficiency and/or effective DBP control. A detailed discussion on multiple disinfectants, including chlorine combinations, is provided in Chapter 9.

As described earlier, the 1995 Community Water System Survey (USEPA, 1997a), indicated that the majority of all surface water and ground water systems in the United States use chlorine for disinfection.

Chlorine has many attractive features that contribute to its wide use in the industry. Four of the key attributes of chlorine are that it:

- Effectively inactivates a wide range of pathogens commonly found in water;
- Leaves a residual in the water that is easily measured and controlled;
- Is economical; and
- Has an extensive track record of successful use in improving water treatment operations (despite the dangers associated with chlorine application and handling, specifically chlorine gas, it still maintains an excellent safety record).

There are, however, some concerns regarding chlorine usage that may impact its uses such as:

- Chlorine reacts with many naturally occurring organic and inorganic compounds in water to produce undesirable DBPs;
- Hazards associated with using chlorine, specifically chlorine gas, require special treatment and response programs; and
- High chlorine doses can cause taste and odor problems.

Chlorination is used in water treatment facilities primarily for disinfection. Because of chlorine's oxidizing powers, it has been found to serve other useful purposes in water treatment, such as (White, 1992):

- Taste and odor control;
- Prevention of algal growths;
- Maintenance of clear filter media;
- Removal of iron and manganese;
- Destruction of hydrogen sulfide;
- Bleaching of certain organic colors;
- Maintenance of distribution system water quality by controlling slime growth;
- Restoration and preservation of pipeline capacity;
- Restoration of well capacity, water main sterilization; and
- Improved coagulation by activated silica.

## 2.7.1 Chlorine Chemistry

Chlorine for disinfection typically is used in one of three forms: chlorine gas, sodium hypochlorite, or calcium hypochlorite. A brief description of the chemistry of these three chemicals is provided below.

### *2.7.1.1 Chlorine Gas*

Chlorine gas hydrolyzes rapidly in water to form hypochlorous acid (HOCl). The following equation presents the hydrolysis reaction:

$$Cl_{2(g)} + H_2O \Rightarrow HOCl + H^+ Cl^- \qquad \textit{Equation 1}$$

Note that the addition of chlorine gas to water reduces the pH of the water due to the production of hydrogen ion.

Hypochlorous acid is a weak acid ($pK_a$ of about 7.5), meaning it dissociates slightly into hydrogen and hypochlorite ions as noted in Equation 2:

$$HOCl \; \hat{U} \; H^+ + O\,Cl^- \qquad \textit{Equation 2}$$

Between a pH of 6.5 and 8.5 this dissociation is incomplete and both HOCl and $OCl^-$ species are present to some extent (White, 1992). Below a pH of 6.5, no dissociation of HOCl occurs, while above a pH of 8.5, complete dissociation to $OCl^-$ occurs. As the germicidal effects of HOCl is much higher than that of $OCl^-$, chlorination at a lower pH is preferred.

#### 2.7.1.2 Hypochlorite

In addition to chlorine gas, chlorine is also available in hypochlorite form as both aqueous solutions and dry solids. The most common aqueous hypochlorite solution is sodium hypochlorite. The most common form of dry solid hypochlorite is calcium hypochlorite (White, 1992).

**Sodium Hypochlorite.** Sodium hypochlorite is produced when chlorine gas is dissolved in a sodium hydroxide solution. Sodium hypochlorite solution typically contains 12.5 percent available chlorine (White, 1992). One gallon of 12.5 percent sodium hypochlorite solution typically contains the equivalent of one pound of chlorine.

The reaction between sodium hypochlorite and water is shown in the following reaction:

$$NaOCl + H_2O \Rightarrow HOCl + Na^+ + OH^- \qquad \textit{Equation 3}$$

Equation 3 shows that the application of sodium hypochlorite to water produces hypochlorous acid, similar to chlorine gas hydrolysis (Equation 1). However, unlike chlorine hydrolysis, the addition of sodium hypochlorite to water yields a hydroxyl ion that will increase the pH of the water. In addition, excess sodium hydroxide is used to manufacture sodium hypochlorite, which will further increase the pH of the water.

**Calcium Hypochlorite.** Calcium hypochlorite is formed from the precipitate that results from dissolving chlorine gas in a solution of calcium oxide (lime) and sodium hydroxide. Granular calcium hypochlorite commercially available typically contains 65 percent available chlorine. This means that 1.5 pounds of calcium hypochlorite contains the equivalent of one pound of chlorine. The reaction between calcium hypochlorite and water is shown in the following reaction:

$$Ca(OCl)_2 + 2H_2O \Rightarrow 2HOCl + Ca^{++} + 2OH^- \qquad \textit{Equation 4}$$

Equation 4 shows that the application of calcium hypochlorite to water also produces hypochlorous acid, similar to chlorine gas hydrolysis (Equation 1). Similar to sodium hypochlorite solution, the addition of calcium hypochlorite to water yields hydroxyl ions that will increase the pH of the water.

## 2.7.2 Chlorine Generation

Onsite generation of chlorine has recently become practical. These generation systems, using only salt and electric power, can be designed to meet disinfection and residual standards and to operate unattended at remote sites. Considerations for chlorine generation include cost, concentration of the brine produced, availability of raw materials, and the reliability of the process (AWWA and ASCE, 1997).

#### 2.7.2.1 Chlorine

Chlorine gas can be generated by a number of processes including the electrolysis of alkaline brine or hydrochloric acid, the reaction between sodium chloride and nitric acid, or the oxidation of

hydrochloric acid. About 70 percent of the chlorine produced in the United States is manufactured from the electrolysis of salt brine and caustic solutions in a diaphragm cell (White, 1992). Since chlorine is a stable compound, it is typically produced off-site by a chemical manufacturer. Once produced, chlorine is packaged as a liquefied gas under pressure for delivery to the site in railcars, tanker trucks, or cylinders.

#### *2.7.2.2 Sodium Hypochlorite*

Dilute sodium hypochlorite solutions (less than 1 percent) can be generated electrochemically on-site from salt brine solution. Typically, sodium hypochlorite solutions are referred to as liquid bleach or Javelle water. Generally, the commercial or industrial grade solutions produced have hypochlorite strengths of 10 to 16 percent. The stability of sodium hypochlorite solution depends on the hypochlorite concentration, the storage temperature, the length of storage (time), the impurities of the solution, and exposure to light. Decomposition of hypochlorite over time can affect the feed rate and dosage, as well as produce undesirable byproducts such as chlorite ions or chlorate (Gordon et al., 1995). Because of the storage problems, many systems are investigating onsite generation of hypochlorite in lieu of its purchase from a manufacturer or vendor (USEPA, 1998b).

#### *2.7.2.3 Calcium Hypochlorite*

To produce calcium hypochlorite, hypochlorous acid is made by adding chlorine monoxide to water and then neutralizing it with a lime slurry to create a solution of calcium hypochlorite. The water is removed from the solution, leaving granulated calcium hypochlorite. Generally, the final product contains up to 70 percent available chlorine and 4 to 6 percent lime. Storage of calcium hypochlorite is a major safety consideration. It should never be stored where it is subject to heat or allowed to contact any organic material of an easily oxidized nature (USEPA, 1998b).

### 2.7.3 Primary Uses and Points of Application of Chlorine

#### *2.7.3.1 Uses*

The main usage of chlorine in drinking water treatment is for disinfection. However, chlorine has also found application for a variety of other water treatment objectives such as, the control of nuisance organisms, oxidation of taste and odor compounds, oxidation of iron and manganese, color removal, and as a general treatment aid to filtration and sedimentation processes (White, 1992; Connell, 1996; Culp/Wesner/Culp, 1986). Table 2-18 presents a summary of chlorine uses and doses.

**Table 2-18. Chlorine Uses and Doses**

| Application | Typical Dose | Optimal pH | Reaction Time | Effectiveness | Other Considerations |
|---|---|---|---|---|---|
| Iron | 0.62 mg/mg Fe | 7.0 | less than 1 hour | Good | |
| Manganese | 0.77 mg/mg Mn | 7–8<br>9.5 | 1–3 hour<br>minutes | Slow kinetics | Reaction time increases at lower pH |
| Biological growth | 1–2 mg/L | 6–8 | NA | Good | DBP formation |
| Taste/odor | Varies | 6–8 | Varies | Varies | Effectiveness depends on compound |
| Color removal | Varies | 4.0–6.8 | Minutes | Good | DBP formation |
| Zebra mussels | 2–5 mg/L<br>0.2–0.5 mg/L[(a)] | | Shock level<br>Maintenance level | Good | DBP formation |
| Asiatic clams | 0.3–0.5 mg/L[(a)] | | Continuous | Good | DBP formation |

Notes:
[(a)] Residual, not dose

Sources: Adapted in part from White, 1992; Connell, 1996; Culp/Wesner/Culp, 1986.

### 2.7.3.2 Points of Application

At conventional surface water treatment plants, chlorine is typically added for prechlorination at either the raw water intake or flash mixer, for intermediate chlorination ahead of the filters, for postchlorination at the filter clearwell, or for rechlorination of the distribution system (Connell, 1996). Table 2-19 summarizes the typical uses for each point of application.

**Table 2-19. Typical Chlorine Points of Application and Uses**

| Point of Application | Typical Uses |
|---|---|
| Raw Water Intake | Zebra mussel and Asiatic clam control, control biological growth |
| Flash Mixer (prior to sedimentation) | Disinfection, iron and manganese oxidation, taste and odor control, oxidation of hydrogen sulfide |
| Filter Influent | Disinfection, control biological growth in filter, iron and manganese oxidation, taste and odor control, algae control, color removal |
| Filter Clearwell | Disinfection |
| Distribution System | Maintain disinfectant residual |

Sources: Connell, 1996; White, 1992; AWWA, 1990.

### 2.7.3.3 Typical Doses

Table 2-20 shows the typical dosages for the various forms of chlorine. The wide range of chlorine gas dosages most likely represents its use as both an oxidant and a disinfectant. While sodium hypochlorite and calcium hypochlorite can also serve as both an oxidant and a disinfectant, their higher cost may limit their use.

**Table 2-20. Typical Chlorine Dosages at Water Treatment Plants**

| Chlorine Compound | Range of Doses |
|---|---|
| Calcium hypochlorite | 0.5–5 mg/L |
| Sodium hypochlorite | 0.2–2 mg/L |
| Chlorine gas | 1–16 mg/L |

Source: SAIC, 1998, as adapted from EPA's review of public water systems' Initial Sampling Plans which were required by EPA's Information Collection Rule (ICR)

## 2.7.4 Pathogen Inactivation and Disinfection Efficacy

### *2.7.4.1 Inactivation Mechanisms*

Research has shown that chlorine is capable of producing lethal events at or near the cell membrane as well as affecting DNA. In bacteria, chlorine was found to adversely affect cell respiration, transport, and possibly DNA activity (Haas and Engelbrecht, 1980). Chlorination was found to cause an immediate decrease in oxygen utilization in both *Escherichia coli* and *Candida parapsilosis* studies. The results also found that chlorine damages the cell wall membrane, promotes leakage through the cell membrane, and produces lower levels of DNA synthesis for *Escherichia coli, Candida parapsilosis*, and *Mycobacterium fortuitum* bacteria. This study also showed that chlorine inactivation is rapid and does not require bacteria reproduction (Haas and Engelbrecht, 1980). These observations rule out mutation or lesions as the principal inactivation mechanisms since these mechanisms require at least one generation of replication for inactivation to occur.

### *2.7.4.2 Environmental Effects*

Several environmental factors influence the inactivation efficiency of chlorine, including water temperature, pH, contact time, mixing, turbidity, interfering substances, and the concentration of available chlorine. In general, the highest levels of pathogen inactivation are achieved with high chlorine residuals, long contact times, high water temperature, and good mixing, combined with a low pH, low turbidity, and the absence of interfering substances. Of the environmental factors, pH and temperature have the most impact on pathogen inactivation by chlorine. The effect of pH and temperature on pathogen inactivation are discussed below.

**pH.** The germicidal efficiency of hypochlorous acid (HOCl) is much higher than that of the hypochlorite ion ($OCl^-$). The distribution of chlorine species between HOCl and $OCl^-$ is determined by pH, as discussed above. Because HOCl dominates at low pH, chlorination provides more effective disinfection at low pH. At high pH, $OCl^-$ dominates, which causes a decrease in disinfection efficiency.

The inactivation efficiency of gaseous chlorine and hypochlorite is the same at the same pH after chlorine addition. Note, however, that addition of gaseous chlorine will decrease the pH (see Equation 1) while the addition of hypochlorite will increase the pH of the water (see Equation 3 and

Equation 4). Therefore, without pH adjustment to maintain the same treated water pH, gaseous chlorine will have greater disinfection efficiency than hypochlorite.

The impact of pH on chlorine disinfection has been demonstrated in the field. For example, virus inactivation studies have shown that 50 percent more contact time is required at pH 7.0 than at pH 6.0 to achieve comparable levels of inactivation. These studies also demonstrated that a rise in pH from 7.0 to 8.8 or 9.0 requires six times the contact time to achieve the same level of virus inactivation (Culp and Culp, 1974). Although these studies found a decrease in inactivation with increasing pH, some studies have shown the opposite effect. A 1972 study reported that viruses were more sensitive to free chlorine at high pH than at low pH (Scarpino et al., 1972).

**Temperature.** For typical drinking water treatment temperatures, pathogen inactivation increases with temperature. Virus studies indicate that the contact time should be increased by two to three times to achieve comparable inactivation levels when the water temperature is lowered by 10°C (Clarke et al., 1962).

### *2.7.4.3 Disinfection Efficacy*

Since its introduction, numerous investigations have been made to determine the germicidal efficiency of chlorine. Although there are widespread differences in the susceptibility of various pathogens, the general order of increasing chlorine disinfection difficulty are bacteria, viruses, and then protozoa.

**Bacteria Inactivation.** Chlorine is an extremely effective disinfectant for inactivating bacteria. A study conducted during the 1940s investigated the inactivation levels as a function of time for *E. coli*, *Pseudomonas aeruginosa*, *Salmonella typhi*, and *Shigella dysenteriae* (Butterfield et al., 1943). Study results indicated that HOCl is more effective than $OCl^-$ for inactivation of these bacteria. These results have been confirmed by several researchers that concluded that HOCl is 70 to 80 times more effective than $OCl^-$ for inactivating bacteria. (Culp/Wesner/Culp, 1986).

**Virus Inactivation.** Chlorine has been shown to be a highly effective viricide. One of the most comprehensive virus studies was conducted in 1971 using treated Potomac estuary water (Liu et al., 1971). The tests were performed to determine the resistance of 20 different enteric viruses to free chlorine under constant conditions of 0.5 mg/L free chlorine and a pH and temperature of 7.8 and 2°C, respectively. In this study, the least resistant virus was found to be reovirus and required 2.7 minutes to achieve 99.99 percent inactivation (4 log removal). The most resistant virus was found to be a poliovirus, which required more than 60 minutes for 99.99 inactivation. The corresponding CT range required to achieve 99.99 percent inactivation for all 20 viruses was between 1.4 to over 30 mg·min/L.

Virus survival studies have also been conducted on a variety of both laboratory and field strains (AWWA, 1979). All of the virus inactivation tests in this study were performed at a free chlorine residual of 0.4 mg/L, a pH of 7.0, a temperature of 5°C, and contact times of either 10, 100, or 1,000 minutes. Test results showed that of the twenty cultures tested only two poliovirus strains reached

99.99 percent inactivation after 10 minutes (CT = 4 mg·min/L), six poliovirus strains reached 99.99 percent inactivation after 100 minutes (CT = 40 mg·min/L), and 11 of the 12 polioviruses plus one *Coxsackievirus* strain (12 out of a total of 20 viruses) reached 99.99 percent inactivation after 1,000 minutes (CT = 400 mg·min/L).

**Protozoa Inactivation.** Chlorine has been shown to have limited success inactivating protozoa. Data obtained during a 1984 study indicated that the resistance of *Giardia* cysts are two orders of magnitude higher than that of enteroviruses and more than three orders of magnitude higher than the enteric bacteria (Hoff et al., 1984). CT requirements for *Giardia* cysts inactivation when using chlorine as a disinfectant has been determined for various pH and temperature conditions (AWWA, 1991). These CT values increase at low temperatures and high pH (See also Table 2-13).

Chlorine has little impact on the viability of *Cryptosporidium* oocysts when used at the relatively low doses encountered in water treatment (e.g., 5 mg/L). Approximately 40 percent removals (0.2 log) of *Cryptosporidium* were achieved at CT values of both 30 and 3,600 mg·min/L (Finch et al., 1994). Another study determined that "no practical inactivation was observed" when oocysts were exposed to free chlorine concentrations ranging from 5 to 80 mg/L at pH 8, a temperature of 22°C, and contact times of 48 to 245 minutes (Gyürék et al., 1996). CT values ranging from 3,000 to 4,000 mg·min/L were required to achieve 1-log of *Cryptosporidium* inactivation at pH 6.0 and temperature of 22°C. During this study, one trial in which oocysts were exposed to 80 mg/L of free chlorine for 120 minutes was found to produce greater than 3-logs of inactivation.

### *2.7.4.4 CT Curves*

Chlorine is regarded as a strong disinfectant that is effective at inactivating bacteria and viruses, and under certain circumstances, *Giardia*. Because of chlorine's extremely high virus inactivation efficiency, CT values are almost always governed by protozoa inactivation. For example, Figure 2-1 shows the CT values required to achieve between 0.5 and 3-logs of virus and *Giardia* inactivation (AWWA, 1991). As shown, the CT values required to achieve the recommended disinfection efficiency for conventional filtration systems (i.e., 0.5-log *Giardia* cyst and 2-log virus inactivation level) are 23 and 3 mg min/L, respectively.

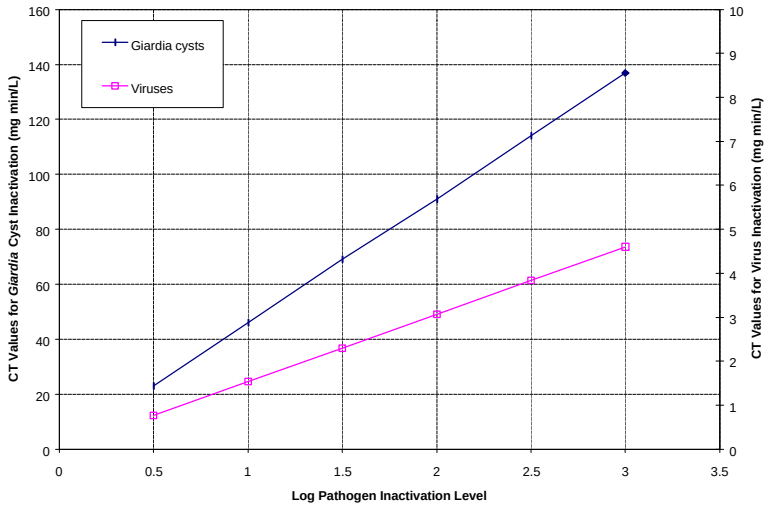


**Figure 2-1. Free Chlorine *Giardia* and Virus CT Requirements**

CT values for *Giardia* inactivation for various pH values and temperatures at a chlorine dose of 3.0 mg/L are shown in Figures 2-2 and 2-3. As shown, the inactivation efficacy of free chlorine decreases with increasing pH and/or decreasing temperature. CT values shown in Figures 2-2 and 2-3 are based on animal infectivity and excystation studies. CT values ranging from 0.5 to 3-log inactivation at temperatures of 0.5 and 5°C were based on a multiplicative model, and applying first order kinetics to the 99 percent upper confidence interval of the 99.99-percentile CT values. CT values for temperatures above 5°C were estimated by assuming a twofold decrease for every 10°C decrease in temperature.

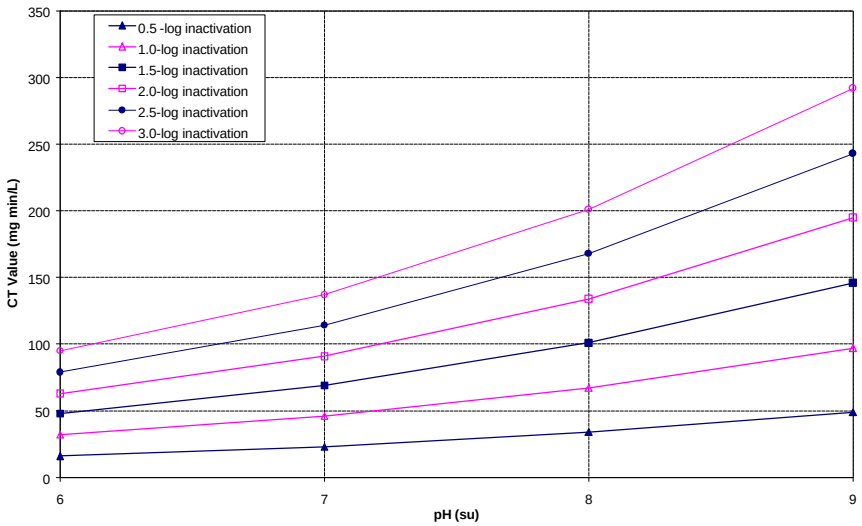


**Figure 2-2. CT Values for Inactivation of *Giardia* Cysts by Free Chlorine at 10°C (at $Cl_2$ dose of 3.0 mg/L)**

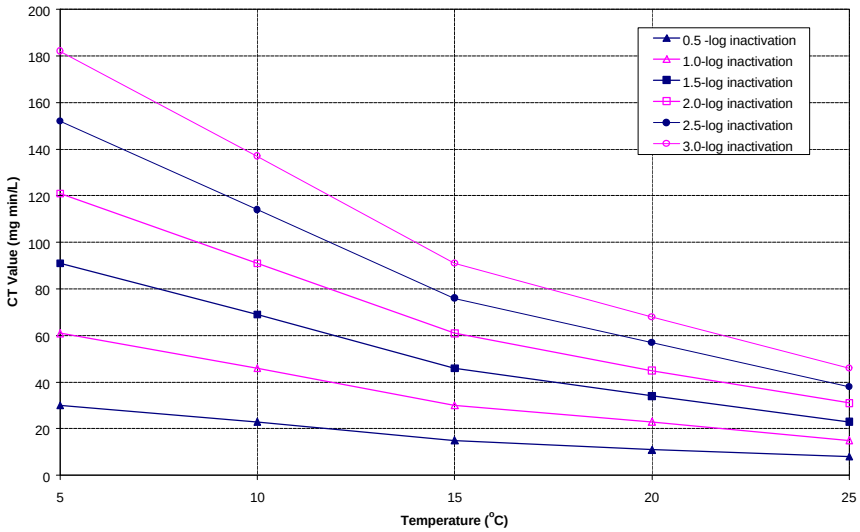


**Figure 2-3. CT Values for Inactivation of *Giardia* Cysts by Free Chlorine at pH 7.0 (at $Cl_2$ dose of 3.0 mg/L)**

## 2.7.5 DBP Formation and Control

### *2.7.5.1 DBP Formation*

Halogenated organics are formed when natural organic matter (NOM) reacts with free chlorine or free bromine. Free chlorine is normally introduced into water directly as a primary or secondary disinfectant. Free bromine results from the oxidation by chlorine of the bromide ion in the source water. Factors affecting the formation of these halogenated DBPs include type and concentration of NOM, chlorine form and dose, time, bromide ion concentration, pH, organic nitrogen concentration, and temperature. Organic nitrogen significantly influenced the formation of nitrogen containing DBPs, including haloacetonitriles, halopicrines, and cyanogen halides (Reckhow et al., 1990; Hoigné and Bader, 1988).

The formation of DBPs is strongly related to TOC at the point of disinfection. DBP formation also correlates with the amount of chlorine consumed (Singer et al., 1995). Stevens et al. (1989) found that higher TTHM formation occurs at high pH (9.4) than at low pH (5.0) while HAA showed no clear trend as a function of pH. A survey of 35 water utilities conducted by MWDSC (Krasner et al., 1989) showed the median TTHM and HAA concentrations measured as 39 and 19 µg/L (i.e. more THMs than HAAs are formed). However, a subsequent study by Singer et al. (1995) found a reversal in dominance, with more HAAs than THMs produced in waters from North Carolina utilities. They postulated that the reason for this change is due to the lower pH levels and differences in TOC and bromide concentrations in the North Carolina waters. Pourmoghaddas et al. (1993) showed that brominated and mixed brominated/chlorinated THMs and HAAs are formed when using chlorine in the presence of bromide.

The occurrence of THMs and HAAs is important because regulatory limits are placed on both groups of compounds. One water utility may therefore find that its chlorination practice is limited by the production of THMs, while another will find that HAAs limit the use of chlorine. This distribution of THMs and HAA is a function of the TOC and bromide concentration in the water, as well as the pH during chlorination.

Of note, Chlorate is produced as a byproduct when hypochlorite degrades during storage.

### 2.7.5.2 DBP Control

DBPs can be controlled by several means, including removing the DBP precursors, modifying the chlorination strategy, changing disinfectants, or removing the DBP itself. Because DBPs are difficult to remove once they are formed, control strategies typically focus on the first three methods.

Studies have shown that removal of TTHM precursors tends to remove the formation potential for the other DBPs. Generally, aggregate DBP formation will decrease as the removal of TOC increases. Recent research indicates that moving the point of chlorination back into the treatment process can reduce the formation of DBPs.

Summers et al. (1997) recently summarized the results from four studies evaluating the impact of pretreatment on DBP formation. Jar tests were conducted to simulate the water treatment through rapid mix, coagulation, flocculation, and sedimentation. Chlorine was added at various points in the jar testing to simulate the impact of various dose points on production of DBPs. The results clearly demonstrate the benefits of delaying the point of chlorination downstream in the treatment train to take advantage of precursor removal during initial flocculation and sedimentation processes. Table 2-21 summarizes the results from this study.

**Table 2-21. Percent Reduction in DBP Formation by Moving Chlorination Point Later In Treatment Train**

| Chlorination point | TTHM Baseline (%) | TTHM Enhanced (%) | HAA5 Baseline (%) | HAA5 Enhanced (%) |
|---|---|---|---|---|
| Pre rapid mix | Basis | 17 | Basis | 4.7 |
| Post rapid mix | 1.6 | 21 | 5.3 | 21 |
| Mid flocculation | 8.7 | 36 | 14 | 36 |
| Post sedimentation | 21 | 48 | 35 | 61 |

Notes: Source: USEPA, 1997b based on Summers et al., 1997.
Baseline = Baseline coagulant (alum) dose for optimal turbidity removal (~30 mg/L)
Enhanced = Enhanced coagulant (alum) dose for optimal TOC removal (~52 mg/L)

Table 2-21 also shows the benefit of enhanced coagulation to reduce DBP production. The THM reduction of 21 percent by moving the chlorination point to post sedimentation is more than doubled to 48 percent by enhanced coagulation. The HAA removal increases from 45 to 61 percent under

enhanced coagulation with post sedimentation chlorination. Therefore, DBP control by selecting the optimal dose location and conditions along with enhanced precursor removal can significantly reduce DBP formation at low added cost.

White (1992) suggested that pretreatment goals should include: 1) maximizing THM precursor removal; 2) reducing ammonia-N concentration to 0.10 mg/L; 3) reducing organic-N concentration to 0.05 mg/L; and 4) limiting 15-minute chlorine demand to 0.5 mg/L. These guidelines should improve raw water quality sufficiently to allow the use of the free chlorine residual process without exceeding the EPA MCLs for TTHMs.

## 2.7.6 Operational Considerations

### *2.7.6.1 Application Methods*

Different application methods are used, depending upon the form of chlorine used. The following paragraphs describe the typical application methods for chlorine, sodium hypochlorite, and calcium hypochlorite.

**Chlorine.** Liquefied chlorine gas is typically evaporated to gaseous chlorine prior to metering. The heat required for evaporation can be provided through either a liquid chlorine evaporator or the ambient heat input to the storage container. Once the compressed liquid chlorine is evaporated, chlorine gas is typically fed under vacuum conditions. Either an injector or a vacuum induction mixer usually creates the required vacuum. The injector uses water flowing through a venturi to draw the chlorine gas into a side stream of carrier water to form a concentrated chlorine solution. This solution is then introduced into the process water through a diffuser or mixed with a mechanical mixer. A vacuum induction mixer uses the motive forces of the mixer to create a vacuum and draws the chlorine gas directly into the process water at the mixer.

**Sodium Hypochlorite.** Sodium hypochlorite solutions degrade over time. For example, a 12.5 percent hypochlorite solution will degrade to 10 percent in 30 days under “best case” conditions (White, 1992). Increased temperature, exposure to light, and contact with metals increase the rate of sodium hypochlorite degradation (Connell, 1996).

Sodium hypochlorite solution is typically fed directly into the process water using a type of metering pump. Similar to chlorine solution, sodium hypochlorite is mixed with the process water with either a mechanical mixer or induction mixer. Sodium hypochlorite solution is typically not diluted prior to mixing to reduce scaling problems.

**Calcium Hypochlorite.** Commercial high-level calcium hypochlorite contains at least 70% available chlorine (USEPA, 1991). Under normal storage conditions, calcium hypochlorite loses 3 to 5% of its available chlorine in a year (AWWA and ASCE, 1997). Calcium hypochlorite comes in powder, granular, and compressed tablet forms (USEPA, 1991). Typically, calcium hypochlorite solution is prepared by mixing powdered or granular calcium hypochlorite with a small flow. The highly chlorinated solution is then flow paced into drinking water flow.

#### 2.7.6.2 Safety and Handling Considerations

**Chlorine.** Chlorine gas is a strong oxidizer. The U.S. Department of Transportation classifies chlorine as a poisonous gas (Connell, 1996). Fire codes typically regulate the storage and use of chlorine. In addition, facilities storing more than 2,500 pounds of chlorine are subject to the following two safety programs:

- Process Safety Management standards regulated by the Occupational Safety and Health Administration under 29 CFR 1910.
- The Risk Management Program Rule administered by EPA under Section 112(r) of the Clean Air Act.

All of these regulations (as well as local and state codes and regulations) must be considered during the design and operation of chlorination facilities at a water treatment plant.

**Sodium Hypochlorite.** Sodium hypochlorite solution is a corrosive liquid with an approximate pH of 12 (AWWA, 1990). Therefore, typical precautions for handling corrosive materials such as avoiding contact with metals, including stainless steel, should be used.

Sodium hypochlorite solutions may contain chlorate. Chlorate is formed during the both the manufacturing and storage of sodium hypochlorite (due to degradation of the product). Chlorate formation can be minimized by reducing the degradation of sodium hypochlorite (Gilbert et al., 1995) by limiting storage time, avoid high temperatures and reduce light exposure.

Spill containment must be provided for the sodium hypochlorite storage tanks. Typical spill containment structures include containment for the entire contents of the largest tank (plus freeboard for rainfall or fire sprinklers), no uncontrolled floor drains, and separate containment areas for each incompatible chemical.

**Calcium Hypochlorite.** Calcium hypochlorite is an oxidant and as such should be stored separately from organic materials that can be readily oxidized. It should also be stored away from sources of heat. Improperly stored calcium hypochlorite has caused spontaneous combustion fires (White, 1992).

# 2.8 Summary

## 2.8.1 Advantages and Disadvantages of Chlorine Use

The following list presents selected advantages and disadvantages of using chlorine as a disinfection method for drinking water (Masschelein, 1992; Process Applications, Inc., 1992). Because of the wide variation of system size, water quality, and dosages applied, some of these advantages and disadvantages may not apply to a particular system.

**Advantages**

- Oxidizes soluble iron, manganese, and sulfides
- Enhances color removal
- Enhances taste and odor
- May enhance coagulation and filtration of particulate contaminants
- Is an effective biocide
- Is the easiest and least expensive disinfection method, regardless of system size
- Is the most widely used disinfection method, and therefore, the best known
- Is available as calcium and sodium hypochlorite. Use of these solutions is more advantageous for smaller systems than chlorine gas because they are easier to use, are safer, and need less equipment compared to chlorine gas
- Provides a residual.

**Disadvantages**

- May cause a deterioration in coagulation/filtration of dissolved organic substances
- Forms halogen-substituted byproducts
- Finished water could have taste and odor problems, depending on the water quality and dosage
- Chlorine gas is a hazardous corrosive gas
- Special leak containment and scrubber facilities could be required for chlorine gas
- Typically, sodium and calcium hypochlorite are more expensive than chlorine gas
- Sodium hypochlorite degrades over time and with exposure to light
- Sodium hypochlorite is a corrosive chemical
- Calcium hypochlorite must be stored in a cool, dry place because of its reaction with moisture and heat
- A precipitate may form in a calcium hypochlorite solution because of impurities, therefore, an antiscalant chemical may be needed
- Higher concentrations of hypochlorite solutions are unstable and will produce chlorate as a byproduct
- Is less effective at high pH
- Forms oxygenated byproducts that are biodegradable and which can enhance subsequent biological growth if a chlorine residual is not maintained.

- Release of constituents bound in the distribution system (e.g., arsenic) by changing the redox state.

## 2.8.2 Summary Table

Table 2-22 presents a summary of the considerations for the use of chlorine as a disinfectant.

**Table 2-22. Summary of Chlorine Disinfection**

| Consideration | Description |
|---|---|
| Generation | Chlorination may be performed using chlorine gas or other chlorinated compounds that may be in liquid or solid form. Chlorine gas can be generated by a number of processes including the electrolysis of alkaline brine or hydrochloric acid, the reaction between sodium chloride and nitric acid, or the oxidation of hydrochloric acid. Since chlorine is a stable compound, chlorine gas, sodium hypochlorite, and calcium hypochlorite are typically produced off-site by a chemical manufacturer. |
| Primary uses | The primary use of chlorination is disinfection. Chlorine also serves as an oxidizing agent for taste and odor control, prevention of algal growths, maintaining clear filter media, removal of iron and manganese, destruction of hydrogen sulfide, color removal, maintaining the water quality at the distribution systems, and improving coagulation. |
| Inactivation efficiency | The general order of increasing chlorine disinfection difficulty is bacteria, viruses, and then protozoa. Chlorine is an extremely effective disinfectant for inactivating bacteria and highly effective viricide. However, chlorine is less effective against *Giardia* cysts. *Cryptosporidium* oocysts are highly resistant to chlorine. |
| Byproduct formation | When added to the water, free chlorine reacts with NOM and bromide to form DBPs, primarily THMs, some haloacetic acids (HAAs), and others. |
| Point of application | Raw water storage, precoagulation/post-raw water storage, presedimentation/ postcoagulation, postsedimentation/prefiltration, post filtration (disinfection), or in the distribution system. |
| Special considerations | Because chlorine is such a strong oxidant and extremely corrosive, special storage and handling considerations should be considered in the planning of a water treatment plant. Additionally, health concerns associated with handling and use of chlorine is an important consideration. |

## 2.8.3 Reference for Additional Information on Chlorine

With the focus of this manual on disinfectants other than chlorine, all of chlorine's uses and capabilities are not described here. For more detailed information regarding the use of chlorine in water treatment, refer to the list of references provided below. For complete references, see the References section at the end of this chapter.

- AWWA (1990)
- Connell (1996)
- DeMers and Renner (1992)
- Hazen and Sawyer (1992)
- Hoigné and Bader (1988)
- Sawyer et al. (1994)
- Singer (1988)
- White (1992)

## 2.9 References

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# 3. OZONE

Ozone was first used for drinking water treatment in 1893 in the Netherlands. While being used frequently in Europe for drinking water disinfection and oxidation, it was slow to transfer to the United States. In 1987, the Los Angeles Aqueduct Filtration Plant was placed in service and now treats up to 600 mgd of drinking water. In 1991, approximately 40 water treatment plants each serving more than 10,000 people in the United States utilized ozone (Langlais et al., 1991). This number has grown significantly, with Rice (in press) reporting that as of April 1998, 264 operating plants in the United States use ozone. Most of these facilities are small: 149 plants are below 1 mgd.

Ozone is used in water treatment for disinfection and oxidation. Early application of ozone in the United States was primarily for non-disinfection purposes such as color removal or taste and odor control. However, since the implementation of the SWTR and proposal of the DBP rule, ozone usage for primary disinfection has increased in the United States.

## 3.1 Ozone Chemistry

Ozone exists as a gas at room temperature. The gas is colorless with a pungent odor readily detectable at concentrations as low as 0.02 to 0.05 ppm (by volume), which is below concentrations of health concern. Ozone gas is highly corrosive and toxic.

Ozone is a powerful oxidant, second only to the hydroxyl free radical, among chemicals typically used in water treatment. Therefore, it is capable of oxidizing many organic and inorganic compounds in water. These reactions with organic and inorganic compounds cause an ozone demand in the water treated, which should be satisfied during water ozonation prior to developing a measurable residual.

Ozone is sparingly soluble in water. At 20$^{o}$C, the solubility of 100 percent ozone is only 570 mg/L (Kinman, 1975). While ozone is more soluble than oxygen, chlorine is 12 times more soluble than ozone. Ozone concentrations used in water treatment are typically below 14 percent, which limits the mass transfer driving force of gaseous ozone into the water. Consequently, typical concentrations of ozone found during water treatment range from <0.1 to 1mg/L, although higher concentrations can be attained under optimum conditions.

Basic chemistry research (Hoigné and Bader, 1983a and 1983b; Glaze et al., 1987) has shown that ozone decomposes spontaneously during water treatment by a complex mechanism that involves the generation of hydroxyl free radicals. The hydroxyl free radicals are among the most reactive oxidizing agents in water, with reaction rates on the order of $10^{10}$ - $10^{13}$ $M^{-1}$ $s^{-1}$, approaching the diffusion control rates for solutes such as aromatic hydrocarbons, unsaturated compounds, aliphatic alcohols, and formic acid (Hoigné and Bader, 1976). On the other hand, the half-life of hydroxyl free radicals is on the order of microseconds, therefore concentrations of hydroxyl free radicals can never reach levels above $10^{-12}$ M (Glaze and Kang, 1988).

As shown in Figure 3-1 ozone can react by either or both modes in aqueous solution (Hoigné and Bader, 1977):

- Direct oxidation of compounds by molecular ozone ($O_{3(aq)}$).
- Oxidation of compounds by hydroxyl free radicals produced during the decomposition of ozone.

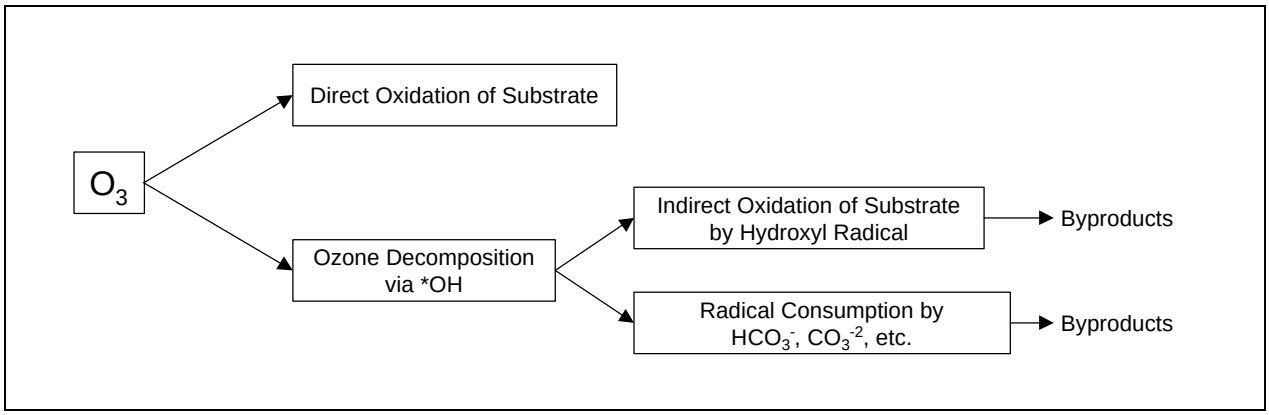



**Figure 3-1. Oxidation Reactions of Compounds (Substrate) During Ozonation of Water**

The two oxidation pathways compete for substrate (i.e., compounds to oxidize). The direct oxidation with aqueous ozone is relatively slow (compared to hydroxyl free radical oxidation) but the concentration of aqueous ozone is relatively high. On the other hand, the hydroxyl radical reaction is fast, but the concentration of hydroxyl radicals under normal ozonation conditions is relatively small. Hoigné and Bader (1977) found that:

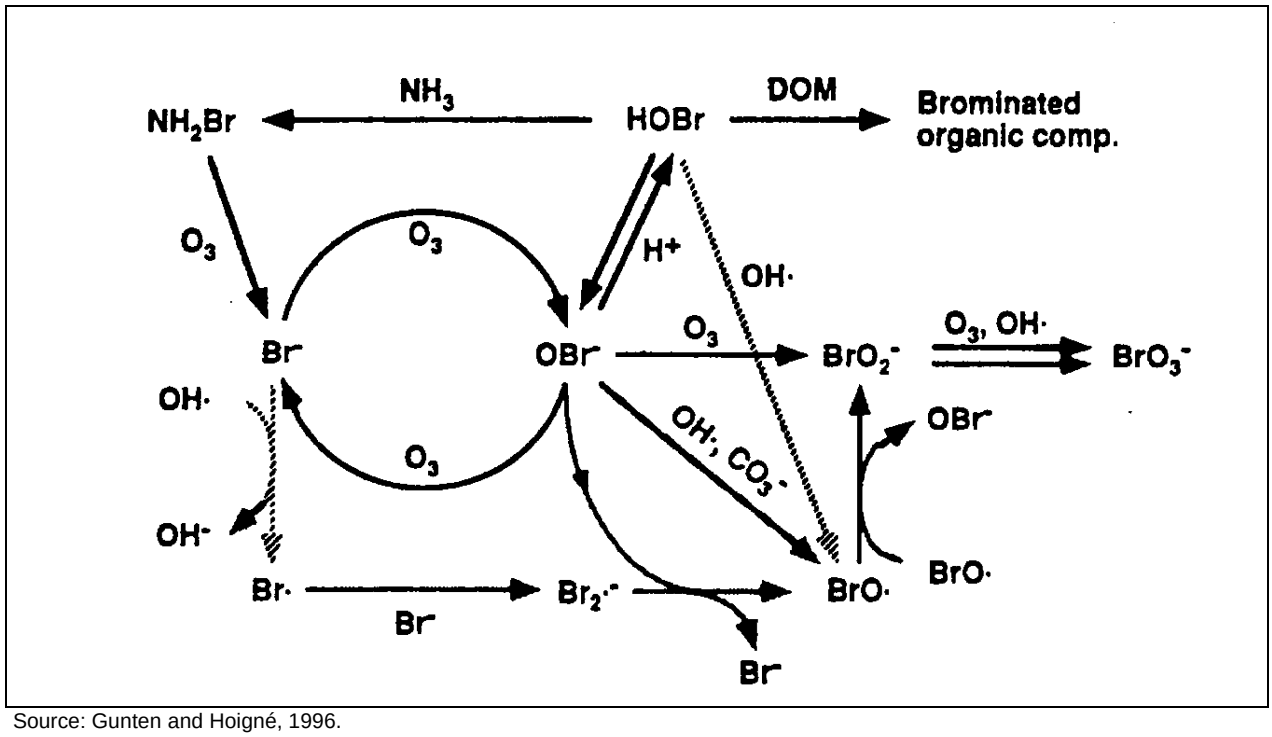
- Under acidic conditions, the direct oxidation with molecular ozone is of primary importance; and
- Under conditions favoring hydroxyl free radical production, such as high pH, exposure to UV, or addition of hydrogen peroxide, the hydroxyl oxidation starts to dominate.

This latter mechanism is used in advanced oxidation processes such as discussed in Chapter 7, Peroxone, to increase the oxidation rates of substrates.

The spontaneous decomposition of ozone occurs through a series of steps. The exact mechanism and reactions associated have not been established, but mechanistic models have been proposed (Hoigné and Bader, 1983a and 1983b; Glaze, 1987). It is believed that hydroxyl radicals forms as one of the intermediate products, and can react directly with compounds in the water. The decomposition of ozone in pure water proceeds with hydroxyl free radicals produced as an intermediate product of ozone decomposition, resulting in the net production of 1.5 mole hydroxyl free radicals per mole ozone.

In the presence of many compounds commonly encountered in water treatment, ozone decomposition forms hydroxyl free radicals. Ozone demands are associated with the following:

- Reactions with natural organic matter (NOM) in the water. The oxidation of NOM leads to the formation of aldehydes, organic acids, and aldo- and ketoacids (Singer, 1992).
- Organic oxidation byproducts. Organic oxidation byproducts are generally more amenable to biological degradation and can be measured as assimilable organic carbon (AOC) or biodegradable dissolved organic carbon (BDOC).
- Synthetic organic compounds (SOCs). Some SOCs can be oxidized and mineralized under favorable conditions. To achieve total mineralization, hydroxyl radical oxidation should usually be the dominant pathway, such as achieved in advanced oxidation processes.
- Oxidation of bromide ion. Oxidation of bromide ion leads to the formation of hypobromous acid, hypobromite ion, bromate ion, brominated organics, and bromamines (see Figure 3-2).
- Bicarbonate or carbonate ions, commonly measured as alkalinity, will scavenge the hydroxyl radicals and form carbonate radicals (Staehelin et al., 1984; Glaze and Kang, 1988). These reactions are of importance for advanced oxidation processes where the radical oxidation pathway is predominant.




Source: Gunten and Hoigné, 1996.

**Figure 3-2. Reaction of Ozone and Bromide Ion Can Produce Bromate Ion and Brominated Organics**

# 3.2 Ozone Generation

## 3.2.1 Ozone Production

Because ozone is an unstable molecule, it should be generated at the point of application for use in water treatment. It is generally formed by, combining an oxygen atom with an oxygen molecule ($O_2$):

$$3O_2 \Leftrightarrow 2O_3$$

This reaction is endothermic and requires a considerable input of energy.

Schönbein (Langlais et < biblio >) first discovered synthetic ozone through the electrolysis of sulfuric acid. Ozone can be produced several ways, although one method, corona discharge, predominates in the ozone generation industry. Ozone can also be produced by irradiating an oxygen-containing gas with ultraviolet light, electrolytic reaction and other emerging technologies as described by Rice (1996).

Corona discharge, also known as silent electrical discharge, consists of passing an oxygen-containing gas through two electrodes separated by a dielectric and a discharge gap. Voltage is applied to the electrodes, causing an electron flowthrough across the discharge gap. These electrons provide the energy to disassociate the oxygen molecules, leading to the formation of ozone. Figure 3-3 shows a basic ozone generator.

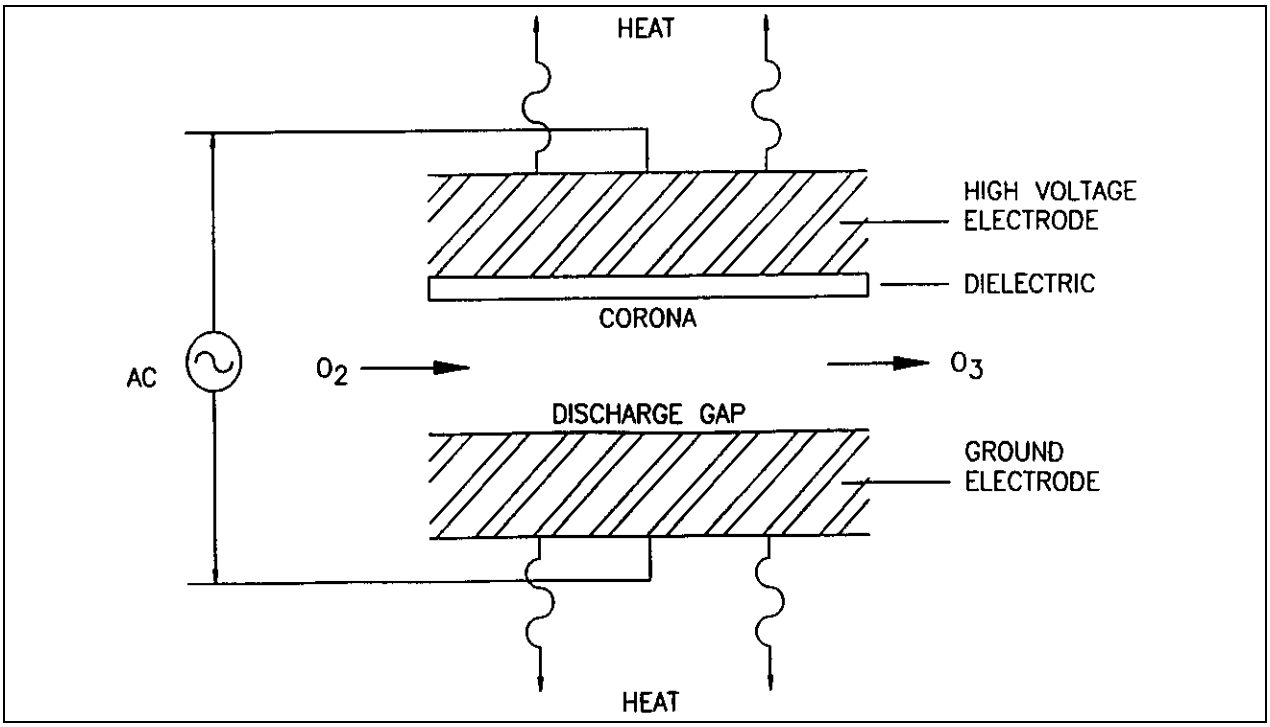



**Figure 3-3. Basic Ozone Generator**

## 3.2.2 System Components

As shown in Figure 3-4, ozone water treatment systems have four basic components: a gas feed system, an ozone generator, an ozone contactor, and an off-gas destruction system. The gas feed system provides a clean, dry source of oxygen to the generator. The ozone contactor transfers the ozone-rich gas into the water to be treated, and provides contact time for disinfection (or other reactions). The final process step, off-gas destruction, is required as ozone is toxic in the concentrations present in the off-gas. Some plants include an off-gas recycle system that returns the ozone-rich off-gas to the first contact chamber to reduce the ozone demand in the subsequent chambers. Some systems also include a quench chamber to remove ozone residual in solution.

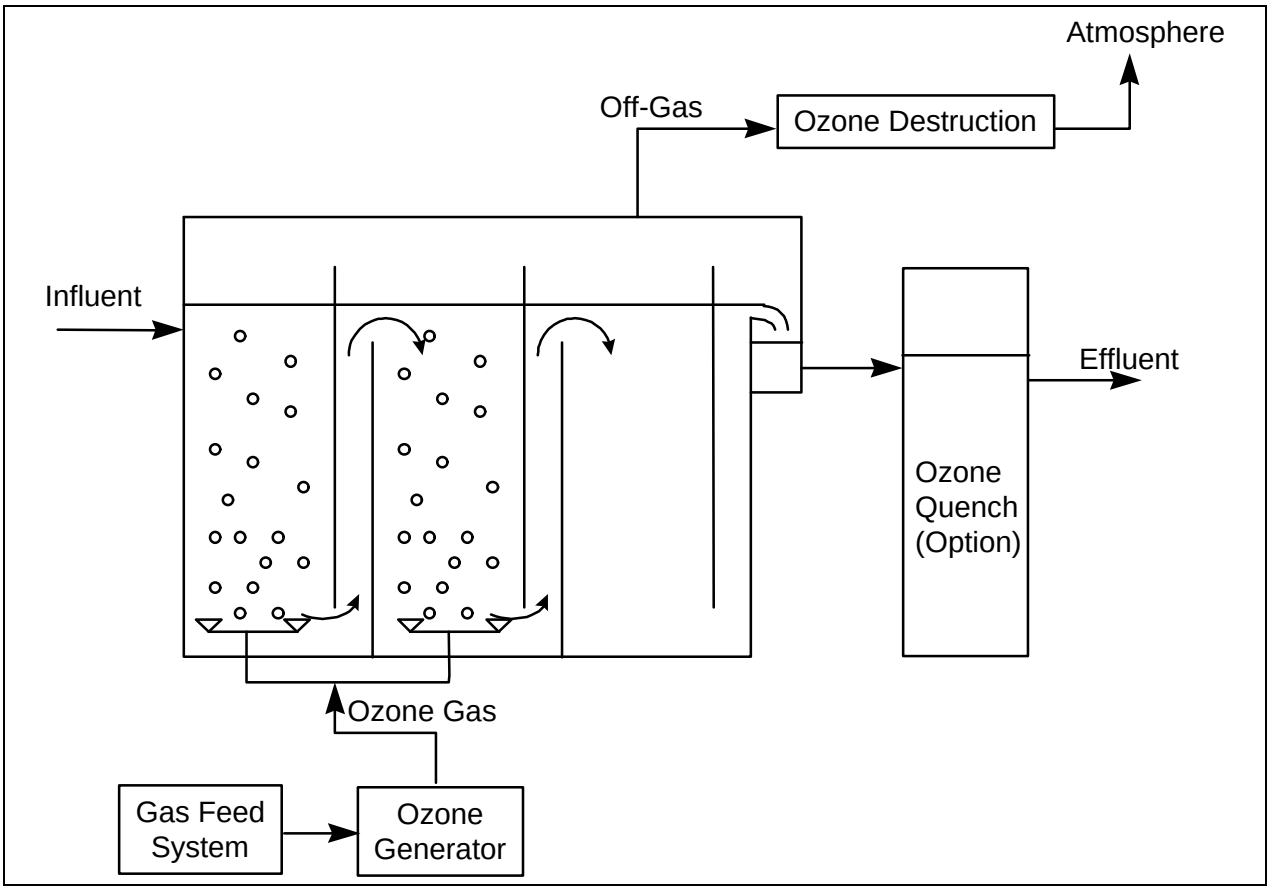



**Figure 3-4. Simplified Ozone System Schematic**

### *3.2.2.1 Gas Feed Systems*

Ozone feed systems are classified as using air, high purity oxygen or mixture of the two. High purity oxygen can be purchased and stored as a liquid (LOX), or it can be generated on-site through either a cryogenic process, with vacuum swing adsorption (VSA), or with pressure swing adsorption (PSA). Cryogenic generation of oxygen is a complicated process and is feasible only in large systems. Pressure swing adsorption is a process whereby a special molecular sieve is used under pressure to

selectively remove nitrogen, carbon dioxide, water vapor, and hydrocarbons from air, producing an oxygen rich (80–95 percent $O_2$) feed gas. The components used in pressure swing adsorption systems are similar to high pressure air feed systems in that both use pressure swing molecular absorption equipment. Low pressure air feed systems use a heat reactivated desiccant dryer.

***Oxygen Feed Systems*** - Liquid oxygen feed systems are relatively simple, consisting of a storage tank or tanks, evaporators to convert the liquid to a gas, filters to remove impurities, and pressure regulators to limit the gas pressure to the ozone generators.

***Air Feed Systems*** - Air feed systems for ozone generators are fairly complicated as the air should be properly conditioned to prevent damage to the generator. Air should be clean and dry, with a maximum dew point of -60º C (-80º F) and free of contaminants. Air preparation systems typically consist of air compressors, filters, dryers, and pressure regulators. Figure 3-5 is a schematic of large scale air preparation system.

Particles greater than 1 μm and oil droplets greater than 0.05 μm should be removed by filtration (Langlais et al., 1991). If hydrocarbons are present in the feed gas, granular activated carbon filters should follow the particulate and oil filters. Moisture removal can be achieved by either compression or cooling (for large-scale system), which lowers the holding capacity of the air, and by desiccant drying, which strips the moisture from the air with a special medium. Desiccant dryers are required for all air preparation systems. Large or small particles and moisture cause arcing which damages generator dielectrics.

Typically, desiccant dryers are supplied with dual towers to allow regeneration of the saturated tower while the other is in service. Moisture is removed from the dryer by either an external heat source or by passing a fraction (10 to 30 percent) of the dried air through the saturated tower at reduced pressure. Formerly, small systems that require only intermittent use of ozone , a single desiccant tower is sufficient, provided that it is sized for regeneration during ozone decomposition time.

Air preparation systems can be classified by the operating pressure: ambient, low (less than 30 psig), medium, and high (greater than 60 psig) pressure. The distinguishing feature between low and high pressure systems is that high pressure systems can use a heatless dryer. A heatless dryer operates normally in the 100 psig range, rather than the 60 psig range. Rotary lobe, centrifugal, rotary screw, liquid ring, vane, and reciprocating compressors can be used in air preparation systems. Table 3-1 lists the characteristics of many of these types of compressors.

Reciprocating and liquid ring compressors are the most common type used in the United States, particularly in small systems, the former because the technology is so prevalent and the latter because liquid ring compressors do not need aftercoolers. Air receivers are commonly used to provide variable air flow from constant volume compressors. Oil-less compressors are used in modern systems to avoid hydrocarbons in the feed gas (Dimitriou, 1990).

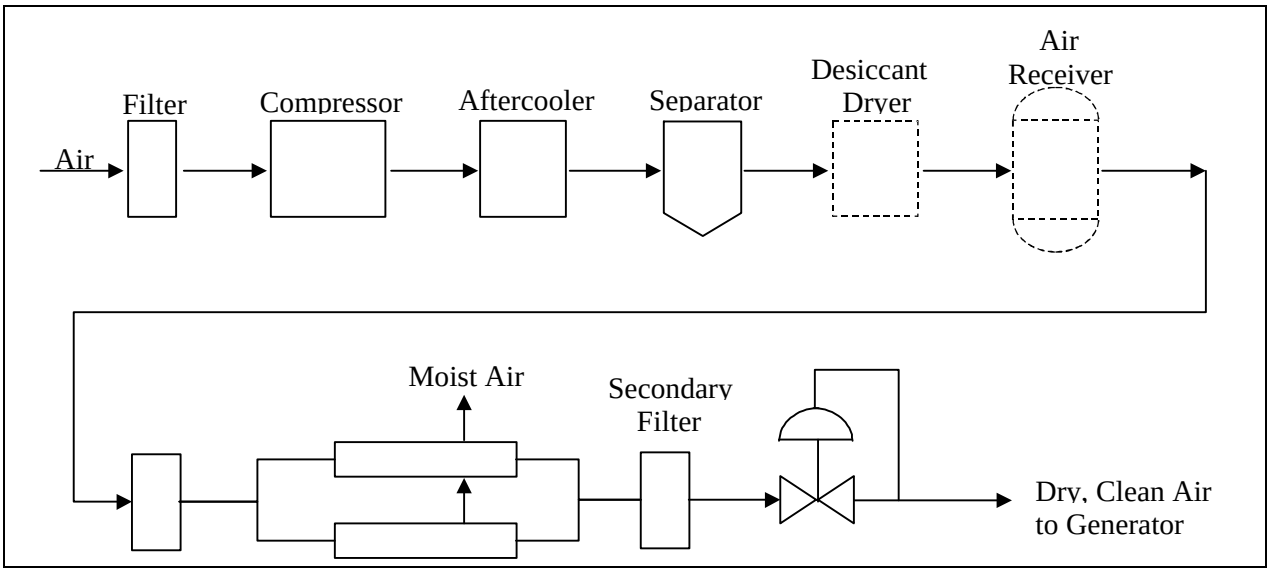



**Figure 3-5. Schematic of an Air Preparation System**

## Table 3-1. Types of Compressors Used in Air Preparation Systems

| Compressor Type | Pressure | Volume | Comments |
|---|---|---|---|
| Rotary lobe | Low – 15 psi | Constant or variable with unloading | Common in Europe |
| Centrifugal | 30 – 100 psi depending on no. of stages | Variable, high volume | Medium efficiency, cost effective in high volumes |
| Rotary Screw | 50 psi (single stage) to 100 psi (2 stage) | Variable with unloading | Slightly more efficient than rotary lobe, draws approximately 40% of full load power in unloaded state, available in non-lubricated design for larger capacities. |
| Liquid Ring | 10–80 psi | Constant volume | Does not require lubrication or aftercooler, relatively inefficient, common in United States. |
| Vane | High - to 100 psi | Constant or variable | Relatively inefficient, not common in U.S. |

Table 3-2 presents a comparison of the advantages and disadvantages of each gas feed system.

**Table 3-2. Comparison of Air and High Purity Oxygen Feed Systems**

| Source | Advantages | Disadvantages |
|---|---|---|
| Air | • Commonly used equipment<br>• Proven technology<br>• Suitable for small and large systems | • More energy consumed per ozone volume produced<br>• Extensive gas handling equipment required<br>• Maximum ozone concentration of 3-5% |
| Oxygen (general) | • Higher ozone concentration (8-14%)<br>• Approximately doubles ozone concentration for same generator<br>• Suitable for small and large systems | • Safety concerns<br>• Oxygen resistant materials required |
| LOX | • Less equipment required<br>• Simple to operate and maintain<br>• Suitable for small and intermediate systems<br>• Can store excess oxygen to meet peak demands | • Variable LOX costs<br>• Storage of oxygen onsite (Fire Codes, i.e. safety concerns)<br>• Loss of LOX in storage when not in use |
| Cryogenic Oxygen Generation | • Equipment similar to air preparation systems<br>• Feasible for large systems<br>• Can store excess oxygen to meet peak demands | • More complex than LOX<br>• Extensive gas handling equipment required<br>• Capital intensive<br>• Complex systems to operate and maintain |

### *3.2.2.2 Ozone Generators*

The voltage required to produce ozone by corona discharge is proportional to the pressure of the source gas in the generator and the width of the discharge gap. Theoretically, the highest yield (ozone produced per unit area of dielectric) would result from a high voltage, a high frequency, a large dielectric constant, and a thin dielectric. However, there are practical limitations to these parameters. As the voltage increases, the electrodes and dielectric materials are more subject to failure. Operating at higher frequencies produces higher concentrations of ozone and more heat requiring increased cooling to prevent ozone decomposition. Thin dielectrics are more susceptible to puncturing during maintenance. The design of any commercial generator requires a balance of ozone yield with operational reliability and reduced maintenance.

Two different geometric configurations for the electrodes are used in commercial ozone generators: concentric cylinders and parallel plates. The parallel plate configuration is commonly used in small generators and can be air cooled. Figure 3-6 shows the basic arrangement for the cylindrical configuration. The glass dielectric/high voltage electrode in commercial generators resembles a fluorescent light bulb and is commonly referred to as a "generator tube."

Most of the electrical energy input to an ozone generator (about 85 percent) is lost as heat (Rice, 1996). Because of the adverse impact of temperature on the production of ozone, adequate cooling should be provided to maintain generator efficiency. Excess heat is removed usually by water

flowing around the stainless steel ground electrodes. The tubes are arranged in either a horizontal or vertical configuration in a stainless steel shell, with cooling water circulating through the shell.

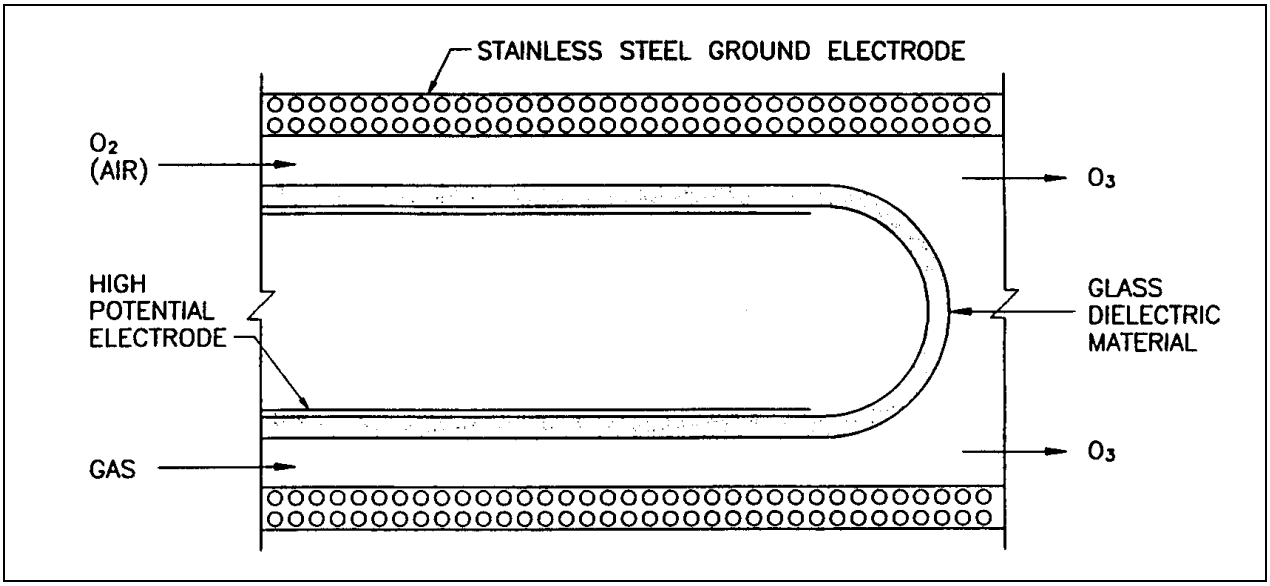



**Figure 3-6. Cylindrical Electrode Schematic**

Ozone generators are classified by the frequency of the power applied to the electrodes. Low frequency (50 or 60 Hz) and medium frequency (60 to 1,000 Hz) generators are the most common found in the water industry, however some high frequency generators are available. Table 3-3 presents a comparison of the three types of generators. Medium frequency generators are efficient and can produce ozone economically at high concentrations, but they generate more heat than low frequency generators and require a more complicated power supply to step up the frequency supplied by utility power. New installations tend to use medium or high frequency generators.

### *3.2.2.3 Ozone Contactors*

Once ozone gas is transferred into water, the dissolved ozone reacts with the organic and inorganic constituents, including any pathogens. Ozone not transferred into the process water during contacting is released from the contactor as off-gas. Transfer efficiencies of greater than 80 percent typically are required for efficient ozone disinfection (DeMers and Renner, 1992).

Common ozone dissolution methods include:

- Bubble diffuser contactors;
- Injectors; and
- Turbine mixers.

## Table 3-3. Comparison of Primary Characteristics of Low, Medium, and High Frequency Ozone Generators

| Characteristic | Low Frequency (50 - 60 Hz) | Medium Frequency (up to 1,000 Hz) | High Frequency (> 1,000 Hz) |
|---|---|---|---|
| Degree of Electronics Sophistication | low | high | high |
| Peak Voltages | 19.5 | 11.5 | 10 |
| Turndown Ratio | 5:1 | 10:1 | 10:1 |
| Cooling Water Required (gal/lb of ozone produced) | 0.5 to 1.0 | 0.5 to 1.5 | 0.25 to 1 |
| Typical Application Range | < 500 lb/day | to 2,000 lb/day | to 2,000 lb/day |
| Operating Concentrations<br>wt - % in air<br>wt - % in oxygen | <br>0.5 to 1.5%<br>2.0 to 5.0% | <br>1.0 to 2.5%+<br>2 to 12% | <br>1.0 to 2.5%+<br>2 to 12% |
| Optimum Ozone Production (as a proportion of total generator capacity) | 60 to 75% | 90 to 95% | 90 to 95% |
| Optimum Cooling Water Differential | 8° to 10°F | 5° to 8°F | 5° to 8°F |
| Power Required (kW-h/lb $O_3$) | air feed: 8 to 12<br>$O_2$ feed: 4 to 6 | air feed: 8 to 12<br>$O_2$ feed: 4 to 6 | air feed: 8 to 12<br>$O_2$ feed: 4 to 6 |
| Air Feed System Power Requirements (kW-h/lb $O_3$) | 5 to 7 | 5 to 7 | 5 to 7 |

Source: Adapted from Rice, 1996, with modifications.

### Bubble Diffuser Contactors

The bubble diffuser contactor is commonly used for ozone contacting in the United States and throughout the world (Langlais et al., 1991). This method offers the advantages of no additional energy requirements, high ozone transfer rates, process flexibility, operational simplicity, and no moving parts. Figure 3-7 illustrates a typical three stage ozone bubble diffuser contactor. This illustration shows a countercurrent flow configuration (ozone and water flowing in opposite directions), an alternating cocurrent/countercurrent arrangement, and a cocurrent flow configuration (ozone and water flowing in the same direction). Also, the number of stages can vary from two to six for ozone disinfection, with the majority of plants using two or three chambers for contacting and reaction (Langlais et al., 1991).

Bubble diffuser contactors are typically constructed with 18 to 22 ft water depths to achieve 85 to 95 percent ozone transfer efficiency. Since all the ozone is not transferred into the water, the contactor chambers are covered to contain the off-gas. Off-gas is routed to an ozone destruct unit, usually catalysts, thermal, or thermal/catalysts.

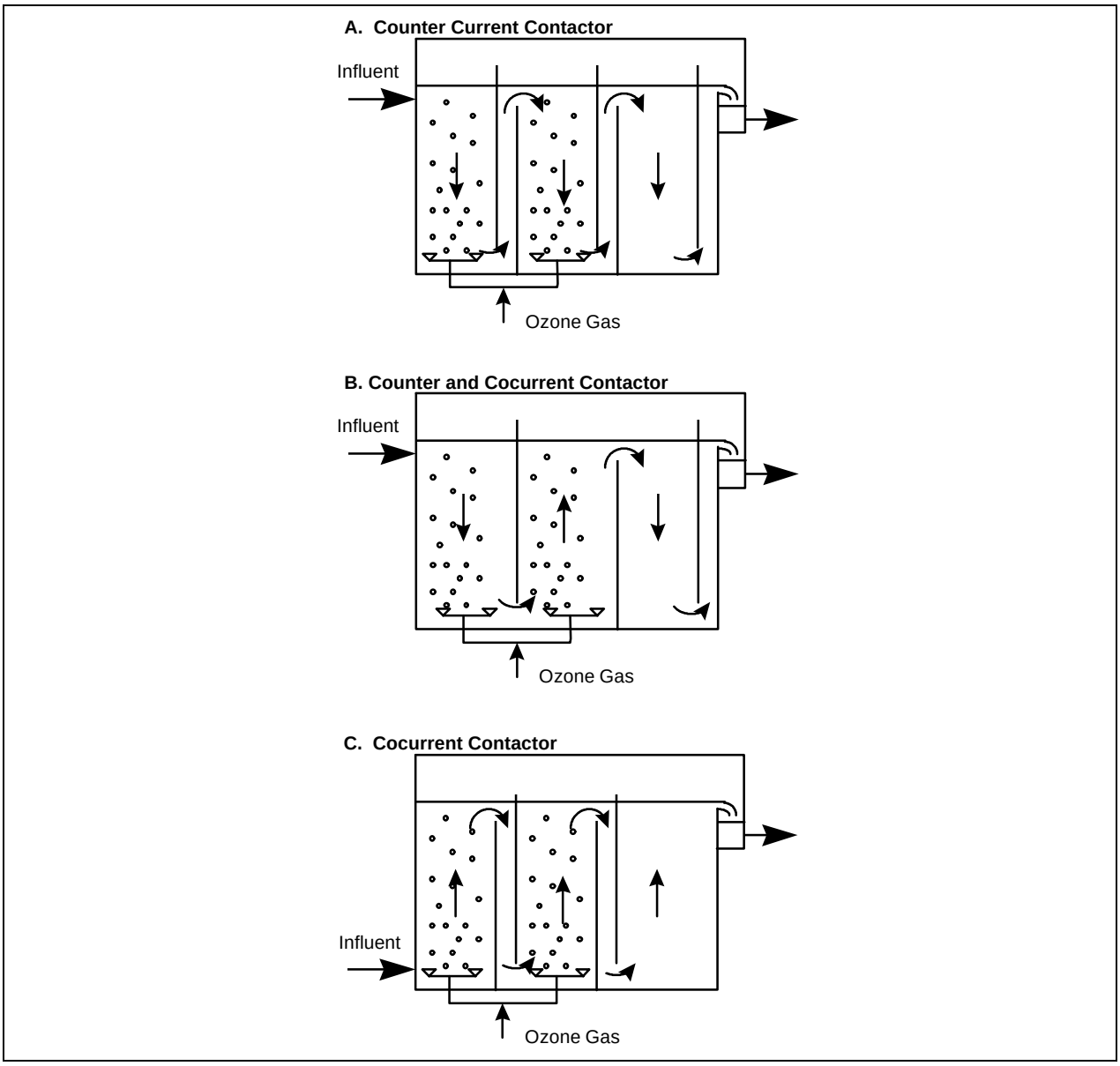



**Figure 3-7. Ozone Bubble Contactor**

Bubble diffuser contactors use ceramic or stainless steel diffusers that are either rod-type or disc-type to generate bubbles. Design considerations for these diffusers (Renner et al., 1988) include:

- Gas flow range of 0.5 to 4.0 scfm;
- Maximum headloss of 0.5 psig;
- Permeability of 2 to 15 cfm/ft$^2$/in of diffuser thickness; and porosity of 35 to 45 percent.

The configuration of the bubble diffuser contactor structure should best be designed to provide plug flow hydraulics. This configuration will minimize the overall volume of the contactor while still

meeting the CT requirements for the system. Contactor volume is determined in conjunction with the applied ozone dosage and estimated residual ozone concentration to satisfy the disinfection CT requirement.

Table 3-4 summarizes the advantages and disadvantages of the bubble diffuser contactor (Langlais et al., 1991). Also, diffuser pore clogging can be a problem when ozone dosages are intermittent and/ or when iron and manganese oxidation is required. Channeling of bubbles is dependent on the type of diffusers used and the spacing between diffusers.

**Table 3-4. Bubble Diffuser Contactor Advantages and Disadvantages**

| Advantages | Disadvantages |
|---|---|
| No moving parts | Deep contact basins |
| Effective ozone transfer | Vertical channeling of bubbles |
| Low hydraulic headloss | Maintenance of gaskets and piping. |
| Operational simplicity | |

### Injector Dissolution

The injector contacting method is commonly used in Europe, Canada, and the United States (Langlais et al., 1991). Ozone is injected into a water stream under negative pressure, which is generated in a venturi section, pulling the ozone into the water stream. In many cases, a sidestream of the total flow is pumped to a higher pressure to increase the available vacuum for ozone injection. After the ozone is injected into this sidestream, the sidestream containing all the added ozone is combined with the remainder of the plant flow under high turbulence to enhance dispersion of ozone into the water. Figure 3-8 illustrates typical in-line and sidestream ozone injection systems.

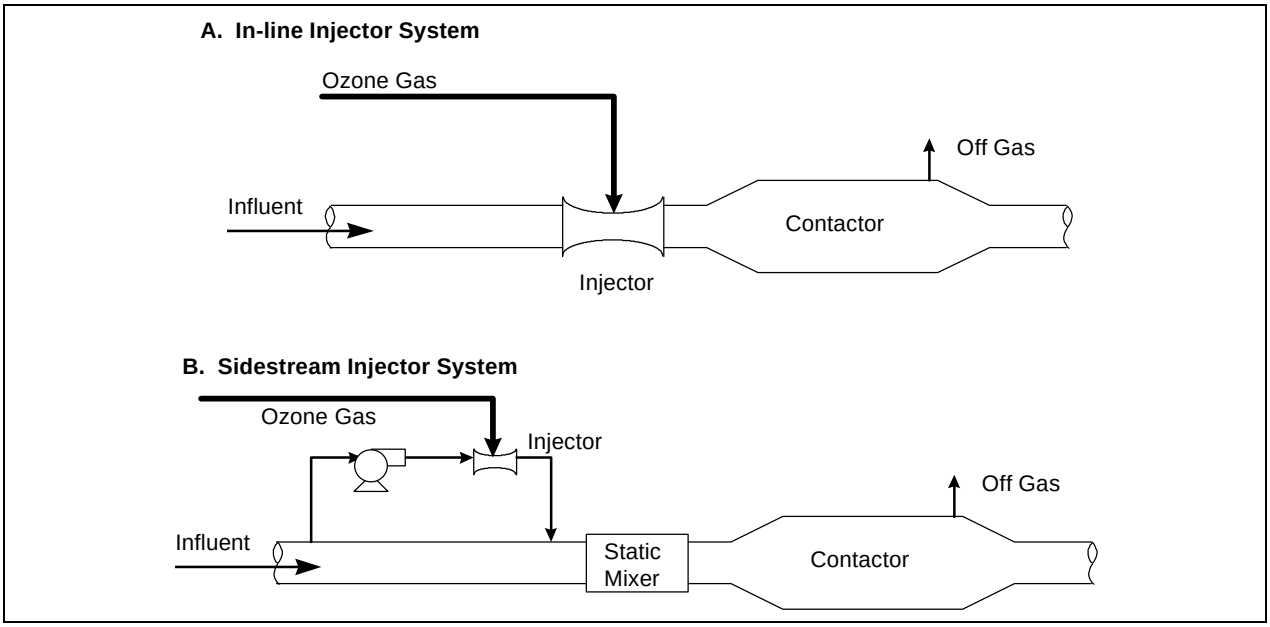



**Figure 3-8. Sidestream Ozone Injection System**

The gas to liquid ratio is a key parameter used in the design of injector contacting systems. This ratio should be less than 0.067 cfm/gpm to optimize ozone transfer efficiency (Langlais et al., 1991). Meeting this criterion typically requires relatively low ozone dosages and ozone gas concentrations greater than 6 percent by weight (DeMers and Renner, 1992). High concentration ozone gas can be generated using a medium-frequency generator and/or liquid oxygen as the feed gas.

To meet the CT disinfection requirements, additional contact time is required after the injector, typically in a plug flow reactor. The additional contact volume is determined in conjunction with the applied ozone dosage and estimated residual ozone concentration to satisfy the disinfection CT requirement.

Table 3-5 summarizes the advantages and disadvantages of injection contacting (Langlais et al., 1991).

**Table 3-5. Injection Contacting Advantages and Disadvantages**

| Advantages | Disadvantages |
|---|---|
| Injection and static mixing have no moving parts | Additional headloss (energy usage) due to static mixers which may require pumping |
| Very effective ozone transfer | Turndown capability limited by injection system |
| Contactor depth less than bubble diffusion | More complex operation and high cost. |

### Turbine Mixer Contactors

Turbine mixers are used to feed ozone gas into a contactor and mix the ozone with the water in the contactor. Figure 3-9 illustrates a typical turbine contactor. The illustrated turbine mixer design shows the motor located outside the basin, allowing for maintenance access. Other designs use a submerged turbine.

Ozone transfer efficiency for turbine mixers can be in excess of 90 percent. However, the power required to achieve this efficiency is 2.2 to 2.7 kW-hr of energy per lb of ozone transferred (Dimitriou, 1990).

Turbine mixing basins vary in water depth from 6 to 15 ft, and dispersion areas vary from 5 to 15 ft (Dimitriou, 1990). Again, as with injector contacting, sufficient contact time may not be available in the turbine basin to meet disinfection CT requirements; consequently additional contact volume may be required.

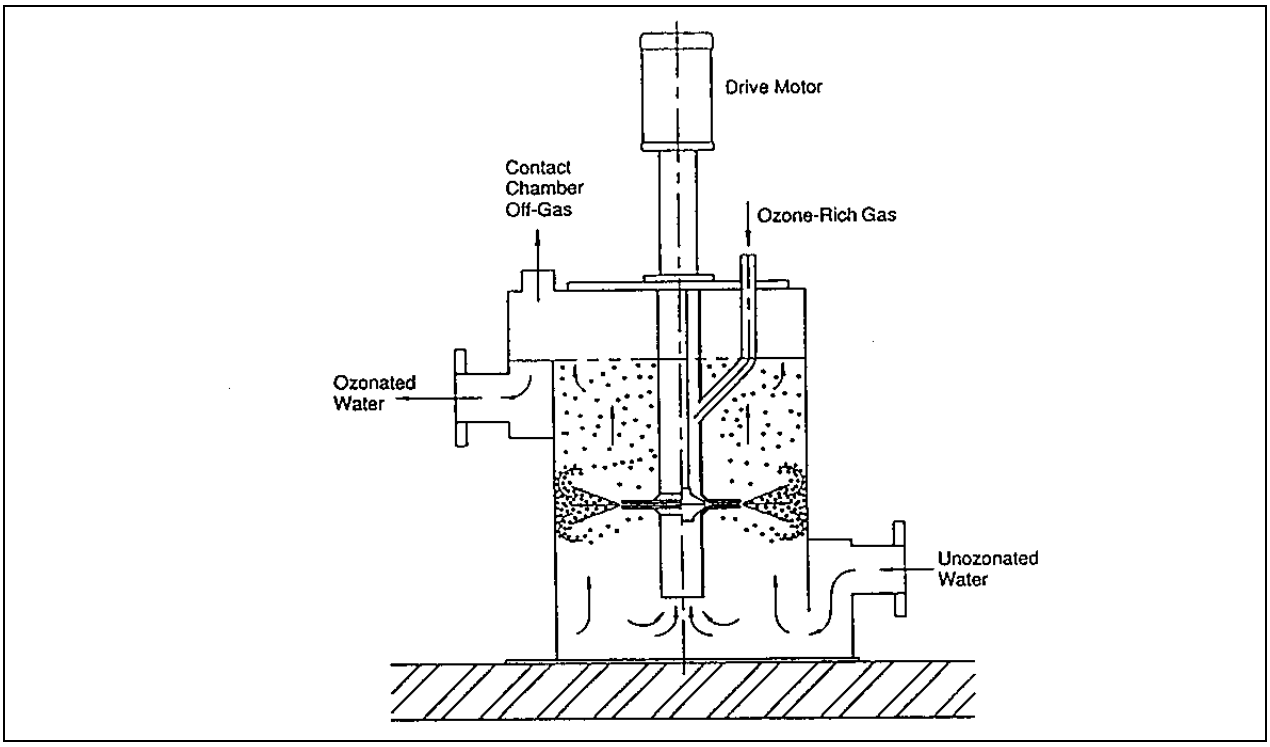



**Figure 3-9. Turbine Mixer Ozone Contactor**

Table 3-6 summarizes advantages and disadvantages for the turbine mixer contactor (Langlais et al., 1991).

**Table 3-6. Turbine Mixer Contactor Advantages and Disadvantages**

| Advantages | Disadvantages |
|---|---|
| Ozone transfer is enhanced by high turbulence resulting in small bubble size | Require energy input |
| Contactor depth less than bubble diffusion | Constant gas flow rate should be maintained, reducing ozone transfer efficiency |
| Aspirating turbines can draw off-gas from other chambers for reuse | Maintenance requirements for turbine and motor |
| Eliminates diffuser clogging concerns | |

### *3.2.2.4 Off-gas Destruction Systems*

The concentration of ozone in the off-gas from a contactor is usually well above the fatal concentration. For example, at 90 percent transfer efficiency, a 3 percent ozone feed stream will still contain 3,000 ppm of ozone in the off-gas. Off-gas is collected and the ozone converted back to oxygen prior to release to the atmosphere. Ozone is readily destroyed at high temperature (> 350° C or by a catalyst operating above 100° C) to prevent moisture buildup. The off-gas destruct unit is designed to reduce the concentration to 0.1 ppm of ozone by volume, the current limit set by OSHA

for worker exposure in an eight hour shift. A blower is used on the discharge side of the destruct unit to pull the air from the contactor, placing the contactor under a slight vacuum to ensure that no ozone escapes.

#### 3.2.2.5 Instrumentation

Instrumentation should be provided for ozone systems to protect both personnel and the equipment. Gas phase ozone detectors should be provided in spaces such as generator rooms where ozone gas may be and personnel are routinely present. An ozone detector is also needed on the outlet from the off-gas destruct unit to ensure the unit is working properly. These units should be interlocked with the ozone generator controls to shut down the ozone generation system should excess ozone be detected. A dew point detector on the feed gas supply just upstream of an ozone generator is required to protect the generator from moisture in the feed gas (when air is the feed gas). Flow switches on the cooling water supply are needed to protect the generator from overheating and a pressure switch to prevent over pressurization.

Other instrumentation can be used to monitor and control the ozone process, although manual control is adequate for small systems, but most small systems are designed to operate automatically, particularly in remote areas. Ozone monitors can be used in conjunction with process flow meters to match ozone dose to process demands and control ozone generation. Sophisticated control schemes can be implemented to minimize the cost of dosing with ozone and reduce operator attention requirements. Many systems include residual monitoring at various points in the contactor to maintain a desired ozone residual and prevent energy-wasting overdosing.

### 3.2.3 Operation and Maintenance

Even though ozone systems are complex, using highly technical instruments, the process is highly automated and very reliable, requiring only a modest degree of operator skill and time to operate an ozone system. Maintenance on generators requires skilled technicians. If trained maintenance staff are not available at the plant, this work can be done by the equipment manufacturer. Some facilities, such as the 600 mgd Los Angeles Aqueduct Filtration Plant, use plant mechanics to perform generator and facilities maintenance. Therefore, backup units are usually installed. Generators should be checked daily when in operation. After a shutdown, dry air or oxygen should be allowed to flow through the generator to ensure that any moisture has been purged prior to energizing the electrodes. At initial start up and after long down times, this process may take up to 12 hours and usually longer when air is the feed gas. As an alternative, a small flow of dry air can be passed through the generator continuously when it is in standby mode to maintain the dry condition.

Filters and desiccant in air preparation systems should be changed periodically, with the frequency depending on the quality of the inlet air and the number of hours in operation. Compressors require periodic service, depending on the type and operating time. LOX tanks should be periodically pressure tested. Piping and contact chambers should be inspected periodically to check for leaks and corrosion.

Dielectric tubes should be periodically cleaned. This operation should be performed when the generator efficiency drops 10-15 percent. Cleaning the tubes is a delicate operation as the tubes are fragile and expensive. Adequate space should be provided for the cleaning operation and for storage of spare tubes.

# 3.3 Primary Uses and Points of Application of Ozone

## 3.3.1 Primary Uses of Ozone

Ozone is used in drinking water treatment for a variety of purposes including:

- Disinfection;
- Inorganic pollutant oxidation, including iron, manganese, and sulfide;
- Organic micropollutant oxidation, including taste and odor compounds, phenolic pollutants, and some pesticides; and
- Organic macropollutant oxidation, including color removal, increasing the biodegradability of organic compounds, DBP precursor control, and reduction of chlorine demand.

### *3.3.1.1 Disinfection*

Ozone is a powerful oxidant able to achieve disinfection with less contact time and concentration than all weaker disinfectants, such as chlorine, chlorine dioxide, and monochloramine (Demers and Renner, 1992). However, ozone can only be used as a primary disinfectant since it cannot maintain a residual in the distribution system. Thus, ozone disinfection should be coupled with a secondary disinfectant, such as chlorine, chloramine, or chlorine dioxide for a complete disinfection system.

### *3.3.1.2 Iron and Manganese Oxidation*

Ozone will oxidize iron and manganese, converting ferrous (2+) iron into the ferric (3+) state and 2+ manganese to the 4+ state. The oxidized forms will precipitate as ferric hydroxide and manganese hydroxide (AWWA, 1990). The precise chemical composition of the precipitate will depend on the nature of the water, temperature, and pH. The ozone dose required for oxidation is 0.43 mg/mg iron and 0.88 mg/mg manganese (Langlais et al., 1991). Iron oxidizes at a pH of 6-9 but manganese is more effective at a pH of around 8. Also, over-ozonation has no effect on iron, but will resolubilize manganese, which then should be reduced to manganese dioxide downstream.

### *3.3.1.3 Oxidation of Taste and Odor Compounds*

Ozone is used to oxidize/destroy taste and odor-causing compounds because many of these compounds are very resistant to oxidation. Suffet et al. (1986) confirmed that ozone is an effective oxidant for use in taste and odor treatment. They found ozone doses of 2.5 to 2.7 mg/L and 10 minutes of contact time (ozone residual of 0.2 mg/L) significantly reduced taste and odors in the

specific waters they tested. Most early U.S. water plants (i.e., 1940-1986) installed ozonation specifically for taste and odor removal.

#### 3.3.1.4 DBP Precursor Control

Early work on oxidation of DBP precursors seemed to indicate that the effects of ozonation, prior to chlorination, were quite site-specific and unpredictable (Umphries et al., 1979). The key variables that seem to determine ozone's effect are dose, pH, alkalinity, and, above all, the nature of the organic material. At low pH levels, precursor destruction by ozone is quite effective; however, above some critical pH, ozone actually is less effective and in fact sometimes increases the amount of chlorination byproduct precursors. For most humic substances this critical pH is 7.5, which is the approximate level at which decomposition of ozone to hydroxyl free radicals increases rapidly, thus increasing organic oxidation rates. Therefore, the implications that at lower pH (approximately 6-7), at which molecular ozone predominates over the hydroxyl free radical, the initial THM precursor by-products are different in nature than those formed by the hydroxyl free radicals oxidized at higher pH levels. This is logical in light of the greater oxidation potential of the hydroxyl free radical over that of ozone.

As alkalinity increases, it has a beneficial effect on THM formation potential (THMFP) (Langlais et al., 1991). This is because alkalinity scavenges any hydroxyl free radicals formed during ozonation, leaving molecular ozone as the sole oxidant, which is only capable of oxidizing organic precursors to a lower oxidation sequence than does the hydroxyl free radical. Given neutral pH and moderate levels of bicarbonate alkalinity, THMFP level reductions of 3 to 20 percent have been shown at ozone doses ranging from 0.2 to 1.6 mg ozone per mg carbon (Singer et al., 1989; Georgeson and Karimi, 1988).

#### 3.3.1.5 Increase Organic Biodegradation

Ozone can be effective in partially oxidizing organics in the water to biodegradable compounds that can be removed by biological filtration (Demers and Renner, 1992). This partial oxidation gives rise to lower molecular weight organics that are more easily biodegradable. This increase in the biodegradable fraction of organic carbon occurs as a result of moderate to high levels of ozonation. These ozone levels are typical of the doses commonly applied for disinfection.

#### 3.3.1.6 Coagulation and Filtration Improvement

Ozone has been reported by some to improve coagulation and filtration efficiency (Gurol and Pidatella, 1993; Farvardin and Collins, 1990; Reckhow et al., 1993; Stolarik and Christie, 1997). However, others have found no improvement in filter effluent turbidity due to ozonation (Tobiason et al., 1992; Hiltebrand et al., 1986). Prendiville (1986) collected data from a large water treatment plant showing that pre-ozonation was more effective than pre-chlorination to reduce filter effluent turbidities. The cause of the improved coagulation is not clear, but several possibilities have been offered (Reckhow et al., 1986), including:

- Oxidation of organic compounds into more polar forms; and
- Oxidation of metals ions to yield insoluble complexes, such as ferric iron complexes.

## 3.3.2 Points of Application

The typical locations for feeding ozone in a water treatment plant are at the head of the treatment plant (raw water) pre-ozonation and after sedimentation.

Raw water quality and turbidity and ozone demand (the amount of ozone required for all oxidation requirements of the water) can be used to assess how to use ozone in the treatment process. Table 3-7 lists the criteria for selecting ozone feed points based on these two parameters. By moving the ozonation process further downstream after sedimentation, the ozone demand and production of byproducts are reduced. The advantage of placing ozone ahead of filtration is that biodegradable organics produced during ozonation can be removed by subsequent biological activity in the filters.

**Table 3-7. Criteria for Selecting Ozone Feed Points for Small Systems**

| Raw Water Quality | Ozone Feed Point(s) | Special Considerations |
|---|---|---|
| **Category I** | | Low ozone demand. |
| Turbidity < 10 NTU | Raw Water or After Sedimentation | Low disinfection byproducts. |
| Ozone Demand < 1mg/L | | Low biodegradable organics. |
| **Category II** | | Low ozone demand. |
| Turbidity > 10 NTU | After Sedimentation | High inorganic particulate. |
| Ozone Demand < 1mg/L | | Low biodegradable organics. |
| **Category III** | | High ozone demand |
| Turbidity < 10 NTU | Raw Water and/or After Sedimentation | Disinfection byproducts |
| Ozone Demand > 1mg/L | | Biodegradable organics formation |
| **Category IV** | | High ozone demand |
| Turbidity > 10 NTU | After Sedimentation and After First | Disinfection byproducts |
| Ozone Demand > 1mg/L | Stage Filtration, if necessary | Biodegradable organics formation |

Source: DeMers and Renner, 1992.

For high quality water with direct filtration, the only practical ozone feed point is the raw water.

Category II (Table 3-7) water is characterized by low ozone demand and high turbidity. This water quality indicates the presence of inorganic material, such as clay or silt particles. For ozone to be most effective for Category II water disinfection, it should be added after either pre-sedimentation or conventional sedimentation.

Raw water with low turbidity and high ozone demand (Category III, Table 3-7) contains dissolved constituents, not suspended, that contribute to a high ozone demand. An example of this type of water is a ground water containing bromide ion, iron, manganese, color, or organics. For this water quality, ozone can be added to either the raw water or after sedimentation. If the water contains organic constituents that become more biodegradable by ozonation, a biological treatment step (see

Section 3.3.4) may be required. The presence of oxidizable organic constituents or bromide ion will generate disinfection byproducts upon ozonation.

Category IV (Table 3-7) water would be considered the most difficult water to treat with ozone due to its high turbidity and high ozone demand. An example of this water quality is surface water containing high concentrations of organic material and inorganic particles. The most effective use of ozone for this water quality is after sedimentation and possibly after filtration. If the water has an extremely high ozone demand, dual ozone feed points may be required to achieve disinfection goals, because the presence of large amounts of organic material may require a biological treatment step and may generate disinfection byproducts.

## 3.3.3 Impact on Other Treatment Processes

Ozonation does have an impact on other processes at the water treatment facility. The impacts of ozone addition include the following:

- The use of ozone generates biodegradable organic matter (BOM) that can result in biological growth which may also increase corrosion rates in distribution systems if not removed by biologically active filtration. When ozonation is placed before biological filters, it can impact the filters by increasing biological growth and increasing backwash frequency.
- Ozone is a strong oxidant that reacts with other oxidants, such as chlorine, chlorine dioxide, and monochloramine.
- Ozone oxidation of iron and manganese generates insoluble oxides that should be removed by sedimentation or filtration. These insoluble oxides also impact the filters by increasing load on the filters and increasing backwash frequency.
- Using pre- and/or internal ozone on most raw waters reduces the subsequent chlorine, chlorine dioxide, or monochloramine demand of the finished water so as to allow a stable chlorine-compound residual to be maintained at a much lower level.

The reader is referred to EPA's *Simultaneous Compliance Guidance Document* (expected to be available in 1999) for additional information regarding the interaction between oxidants and other treatment processes.

## 3.3.4 Biologically Active Filtration

Ozonation typically increases the biodegradability of NOM in water because many large organic molecules are converted into smaller organic molecules that are readily biodegradable. This increase in biodegradable dissolved organic carbon (BDOC) can lead to accelerated bacterial growth and regrowth in the distribution system if not removed in the treatment plant. LeChevallier et al. (1992) found that AOC levels less than 100 ppb may be necessary to control excessive bacterial regrowth in the distribution system if not removed in the treatment plant.

When ozonation is placed upstream of filtration, and environmental conditions such as dissolved oxygen, pH, and temperature are favorable, microbiological activity is increased in the filter and BDOC/AOC removal is enhanced. Ozone addition not only increases the biodegradability of the

dissolved organics, but also introduces large amounts of oxygen to the water, thus, creating an excellent environment for biological growth on the filter media. The advantages of biologically active filtration (Price, 1994) include the following which are all being met in most U.S. plants using ozone:

- Production of a biologically stable water that does not promote excessive bacterial growth and regrowth in the distribution system.
- Removal of NOM that can serve as precursors to byproduct formation as a result of residual disinfection with free or combined chlorine.
- Ozone oxidation as a primary disinfectant prior to biologically active filtration reduces the BDOC concentration in finished water, thus reducing chances of regrowth.
- Reduction of the residual disinfectant demand of the product water so that proposed regulations limiting the maximum disinfectant residual can be met.
- Removal or control of ozonation byproducts.

Biological activity can be supported on slow sand, rapid rate, and GAC media because these media provide a surface for bacteria to attach. Factors such as available surface area, hydraulic loading rate, contact time, availability of nutrients, temperature, and others will determine the performance and BDOC removal efficiency. Biomass develops to higher levels on GAC because of the rougher surface characteristics than on anthracite and sand.

#### 3.3.4.1 Slow Sand Filters

Ozone addition prior to slow sand filtration can increase the efficiency of TOC removal by about 35 percent (Rachwal et al., 1988; Zabel, 1985). Ozone addition can also increase the efficiency of BDOC removal with slow sand filters (Eighmy et al., 1991; Malley et al., 1993).

#### 3.3.4.2 Rapid Rate Filters

Research in the area of biologically active rapid rate filters has focused on the reduction of assimilable organic carbon (AOC) instead of BDOC. While studies have shown rapid rate filtration, employing either sand or dual media lowers AOC levels following ozonation, AOC does not measure all the BDOC. AOC measures only that portion of the BDOC that is more easily assimilable or more easily biodegradable under specific laboratory conditions by two specific microorganisms. Research data shows that biodegradation of AOC can occur in rapid rate filters. The data should be viewed with caution, since the more slowly biodegradable DOC, not measured by AOC, may be passing through rapid rate filters.

#### 3.3.4.3 Granular Activated Carbon

GAC is made biologically active by the deliberate introduction of sufficient dissolved oxygen to water just before passing through GAC columns (Katz, 1980). The high surface area and long retention time in GAC provide an ideal environment to enhance biological growth.

Although ozone actually increases the amount of BDOC, the efficiency of subsequent biodegradation on GAC can be such that the BDOC in GAC effluent is lower than the BDOC in the ozone influent (Langlais et al., 1991). The degree to which biodegradable DOC is removed by ozone/GAC depends upon the process conditions of temperature, amount of BDOC, and the GAC column loading rate, measured by empty bed contact time (EBCT). For example, with an influent BDOC of 0.65 mg C/L and a 10 minute EBCT, one would expect an effluent BDOC of 0.25 mg C/L. The effluent BDOC then could be lowered by either:

- Adding ozone, which would increase the GAC influent BDOC and, therefore, lower the effluent BDOC; or
- Adding more GAC or decreasing the loading rate, which would extend the EBCT and lower the effluent BDOC (Billen et al., 1985, as cited in Langlais et al., 1991).

Huck et al. (1991) reported results from AOC profiles measured in a pilot treatment plant. The plant treated Saskatchewan River water and included coagulation, flocculation, and sedimentation prior to ozonation. Following ozonation, the water was filtered through a dual media (anthracite-sand) filter followed by GAC adsorption. The results demonstrate:

- Variable AOC removal through coagulation, flocculation, and sedimentation (80 percent to zero);
- Increased AOC after ozonation;
- AOC removal through dual media filtration improving at lower hydraulic loading rates and filtered effluent AOC often less than raw water AOC, but highly variable; and
- AOC levels after GAC were low, almost always below raw water AOC concentrations and adsorption appears to contribute to some immediate AOC removal.

## 3.3.5 Pathogen Inactivation and Disinfection Efficacy

Ozone has a high germicidal effectiveness against a wide range of pathogenic organisms including bacteria, protozoa, and viruses. Because of its high germicidal efficiency, ozone can be used to meet high inactivation required by water treatment systems with or without filters. However, ozone cannot be used as a secondary disinfectant because the ozone residual decays too rapidly. The ozone disinfection efficiency is not affected by pH (Morris, 1975), although because of hydroxyl free radicals and rapid decay, efficiency is the same but more ozone should be applied at high pH to maintain “C”.

## 3.3.6 Inactivation Mechanisms

Inactivation of bacteria by ozone is attributed to an oxidation reaction (Bringmann, 1954; Chang, 1971). The first site to be attacked appears to be the bacterial membrane (Giese and Christensen, 1954) either through the glycoproteins or glycolipids (Scott and Lesher, 1963) or through certain amino acids such as typtophan (Goldstein and McDonagh, 1975). In addition, ozone disrupts enzymatic activity of bacteria by acting on the sulfhydryl groups of certain enzymes. Beyond the cell membrane and cell wall, ozone may act on the nuclear material within the cell. Ozone has been found to affect both purines and pyrimidines in nucleic acids (Giese and Christensen, 1954; Scott and Lesher, 1963).

The first site of action for virus inactivation is the virion capsid, particularly its proteins (Cronholm et al., 1976 and Riesser et al., 1976). Ozone appears to modify the viral capsid sites that the virion uses to fix on the cell surfaces. High concentrations of ozone dissociate the capsid completely. One researcher found that the mechanism of ozone inactivation of bacteriophage f2 ribonucleic acid (RNA) included releasing RNA from the phage particles after the phage coat was broken into many pieces (Kim et al., 1980). This finding suggests that ozone breaks the protein capsid, thereby liberating RNA and disrupting adsorption to the host pili. Further, the naked RNA may be secondarily inactivated by ozone at a rate less than that for RNA within the intact phage. The mechanism for inactivation of deoxyribonucleic acid (DNA) bacteriophage T4 has been found to be quite similar to RNA inactivation: ozone attacks the protein capsid, liberates the nucleic acid, and inactivates the DNA (Sproul et al., 1982). In contrast, more recent work on the tobacco mosaic virus (TMV) shows that ozone has a specific effect on RNA. Ozone was found to attack both the protein coat and RNA. The damaged RNA cross-links with amino acids of the coat protein subunits. The authors concluded that TMV loses its infectivity because of its loss of protein coating.

Microscopic observation of inactivation of trophozoites of *Naegleria* and *Acanthamoeba* showed that they were rapidly destroyed and the cell membrane was ruptured (Perrine et al., 1984). Perrine and Langlais showed that ozone affect the plugs in *Naegleria gruberi* cysts (Langlais and Perrine, 1984). Depending on the ozonation conditions, these plugs were completely removed or were partially destroyed. It has been speculated that ozone initially affects the *Giardia muris* cysts wall and makes it more permeable (Wickramanayake, 1984c). Subsequently, aqueous ozone penetrates into the cyst and damages the plasma membranes, additional penetration of ozone eventually affects the nucleus, ribosomes, and other ultrastructural components.

## 3.3.7 Disinfection Parameters

Hoigné and Bader demonstrated that the rate of decomposition of ozone is a complex function of temperature, pH, and concentration of organic solutes and inorganic constituents (Hoigné and Bader, 1975, and 1976). The following sections describe the effects that pH, temperature, and suspended matter have on the reaction rate of ozone and pathogen inactivation.

The ability to maintain a high aqueous ozone concentration is critical from a regulatory disinfection compliance standpoint. This means that factors that accelerate ozone decomposition are undesirable for inactivation because the ozone residual dissipates faster and therefore reduces the CT credit, requiring a corresponding increase in the ozone applied, thus increasing cost.

#### 3.3.7.1 pH

Studies have indicated that pH has little effect on the ability of dissolved ozone residuals to inactivate acid-fast bacteria, such as *Mycobacteria* and *Actinomycetes* (Farooq, 1976). A slight decrease has been found in the virucidal efficacy of ozone residuals as pH decreased (Roy, 1979). However, the opposite effect was observed by Vaughn et al. (1987) (cited in Hoff, 1986). Changes in disinfection efficacy with variations in pH appear to be caused by the ozone decomposition rate. Ozone decomposition occurs faster in higher-pH aqueous solutions and forms various types of oxidants with differing reactivities (Langlais et al., 1991). Tests carried out at constant ozone residual concentration and different pH values showed that the degree of microorganism inactivation remained virtually unchanged (Farooq et al., 1977). More recent studies have indicated decreased virus inactivation by ozone at alkaline pH (pH 8 to 9) for poliovirus 1 (Harakeh and Butler, 1984) and rotaviruses SA-11 and Wa (Vaughn et al., 1987).

Inactivation of *Giardia muris* cysts was found to improve when the pH increased from 7 to 9 (Wickramanayake, 1984a). This phenomenon was attributed to the possible changes in cyst chemistry making it easier for ozone to react with the cyst constituents at the higher pH levels. However, the same study found that inactivation of *Naegleria gruberi* cysts was slower at a pH 9 than at lower pH levels, thereby indicating that pH effects are organism-specific.

#### 3.3.7.2 Temperature

As temperature increases, ozone becomes less soluble and less stable in water (Katzenelson et al., 1974); however, the disinfection and chemical oxidation rates remain relatively stable. Studies have shown that although increasing the temperature from 0 to 30°C can significantly reduce the solubility of ozone and increases its decomposition rate, temperature has virtually no effect on the disinfection rate of bacteria (Kinman, 1975). In other words, the disinfection rate was found to be relatively independent of temperature at typical water treatment plant operating temperatures despite the reduction in solubility and stability at higher temperatures.

#### 3.3.7.3 Suspended Matter

Ozone inactivation of viruses and bacteria contained in aluminum floc (in the size range comparable to those that could typically escape filtration) was not reduced at floc turbidity levels of 1 and 5 NTU (Walsh et al., 1980). This study demonstrated that the microorganisms received no protection from the aluminum floc. Similar results have been obtained for poliovirus 1, coxsackie virus A9, and *E. coli* associated with bentonite clay (Boyce et al., 1981). However, adsorption of the f2 bacteriophage at 1 and 5 NTU of bentonite clay was found to retard the rate of inactivation of ozone (Boyce et al., 1981).

In some instances, river waters heavily polluted with organic matter were ozonated, and the results indicated a degradation of large organic molecules into fragments more easily metabolized by microorganisms. This fragmentation coupled with the inability of ozone to maintain an active concentration in the distribution system, has led to increased slime growth and, consequently, water quality deterioration during distribution (Troyan and Hanson, 1989).

## 3.3.8 Inactivation of Microorganisms

The following sections contain a description of the disinfection efficiency of ozone in terms of bacteria, virus, and protozoa inactivation.

### *3.3.8.1 Bacterial Inactivation*

Ozone is very effective against bacteria. Studies have shown the effect of small concentrations of dissolved ozone (i.e., 0.6 μg/L) on *E. coli*. (Wuhrmann and Meyrath, 1955) and *Legionella pneumophila* (Domingue, et al., 1988). *E. coli.* levels were reduced by 4 logs (99.99 percent removal) in less than 1 minute with a ozone residual of 9 μg/L at a temperature of 12°C. *Legionella pneumophila* levels were reduced by greater than 2 logs (99 percent removal) within a minimum contact time of 5 minutes at a ozone concentration of 0.21 mg/L. Results similar to those obtained for *E. coli.* have been found for *Staphyloccus* sp. and *Pseudomonas fluorescens* inactivation. *Streptococcus faecalis* required a contact time twice as long with the same dissolved ozone concentration, and *Mycobacterium tuberculosis* required a contact time six times as long for the same reduction level as *E. coli*.

In regard to vegetative bacteria, *E. coli* is one of the most sensitive types of bacteria. Furthermore, significant difference has been found among all the Gram-negative bacillae, including *E. coli* and other pathogens such as *Salmonella*, which are all sensitive to ozone inactivation. whereas the Gram-positive cocci (*Staphyloccus* and *Streptococcus*), the Gram-positive bacillae (*Bacillus*), and the *Mycobacteria* are the most resistant forms of bacteria. Sporular bacteria forms are always far more resistant to ozone disinfection than vegetative forms (Bablon, et al., 1991), but all are easily destroyed by relatively low levels of ozone.

### *3.3.8.2 Protozoa Inactivation*

Protozoan cysts are much more resistant to ozone and other disinfectants than vegetative forms of bacteria and viruses. *Giardia lamblia* has a sensitivity to ozone that is similar to the sporular forms of *Mycobacteria*. Both *Naegleria* and *Acanthamoeba* cysts are much more resistant to ozone (and all other disinfectants) than *Giardia* cysts. (Bablon et al., 1991). CT products for 99 percent inactivation of *Giardia lamblia* and *N. gruberi* at 5°C were 0.53 and 4.23 mg • min/L, respectively (Wickramanayake et al., 1984a and 1984b). Data available for inactivation of *Cryptosporidium* oocysts, suggest that among protozoans, this microorganism is more resistant to ozone (Peeters et al., 1989; Langlais et al., 1990). One study found that *Cryptosporidium* oocysts are approximately 10 times more resistant to ozone than *Giardia* (Owens et al., 1994).

### 3.3.8.3 Virus Inactivation

Typically, viruses are more resistant to ozone than vegetative bacteria but less resistant than sporular forms of *Mycobacteria* (Bablon, et al., 1991). The most sensitive forms of viruses are phages, and there seems to be little difference between the polio- and coxsackie viruses. The sensitivity of human rotavirus to ozone was determined to be comparable to that of *Mycobacteria* and polio- and coxsackie viruses (Vaughn et al., 1987).

Keller et al. (1974) studied ozone inactivation of viruses by using both batch tests and pilot plant data. Inactivation of poliovirus 2 and coxsackie virus B3 was more than 3 logs (99.9 percent) in the batch tests with an ozone residual of 0.8 mg/L and 1.7 mg/L and a contact time of 5 minutes. Greater than 5 log (99.999 percent) removal of coxsackie virus was achieved in the pilot plant with an ozone dosage of 1.45 mg/L, which provided an ozone residual of 0.28 mg/L in lake water.

### 3.3.8.4 CT Curves for Giardia Lamblia

CT values shown in Figure 3-9 are based on disinfection studies using in vitro excystation of *Giardia lamblia*. CT values obtained at 5°C and pH 7 were used as the basis for deriving the CT values at other temperatures. A safety factor of 2 has been applied to the values shown in Figure 3-9.

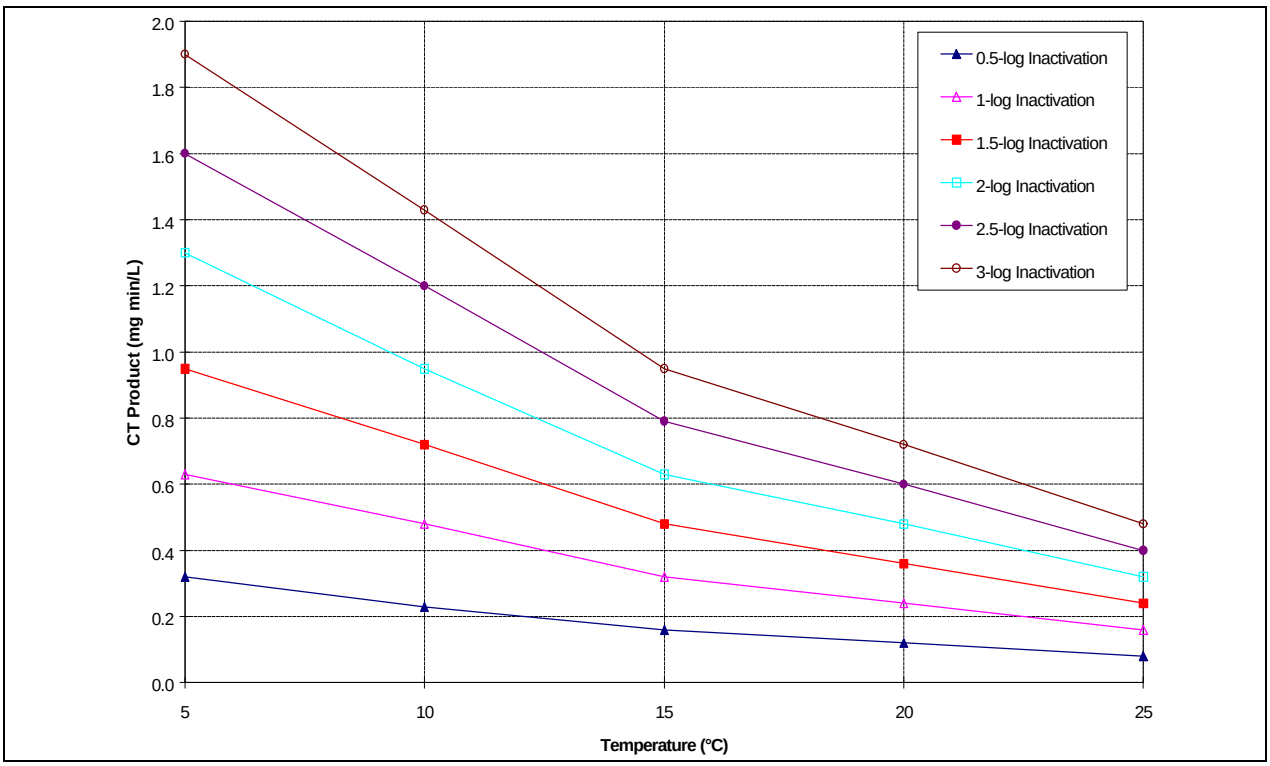



**Figure 3-9. CT Values for Inactivation of Giardia Cysts by Ozone (pH 6 to 9)**

CT values shown in Figure 3-10 for achieving 2-log inactivation of viruses were determined by applying a safety factor of 3 to data obtained from a previous study on poliovirus 1 (Roy et al., 1982). CT values for 3 and 4-log removal were derived by applying first order kinetics and assuming the same safety factor of 3. Data obtained at a pH of 7.2 was assumed to apply for the pH range of 6 to 9.

Several research groups have investigated the efficiency of ozone for *Cryptosporidium* oocyst inactivation. Table 3-8 summarizes CT values obtained for 99 percent inactivation of *Cryptosporidium* oocysts. Results indicate that ozone is one of the most effective disinfectants for controlling *Cryptosporidium* (Finch, et al., 1994) and that *Cryptosporidium muris* may be slightly more resistant to ozonation than *Cryptosporidium parvum* (Owens et al., 1994). A wide range of CT values has been reported for the same inactivation level, primarily because of the different methods of *Cryptosporidium* measurements employed and pH, temperature, and above all, ozonation conditions.

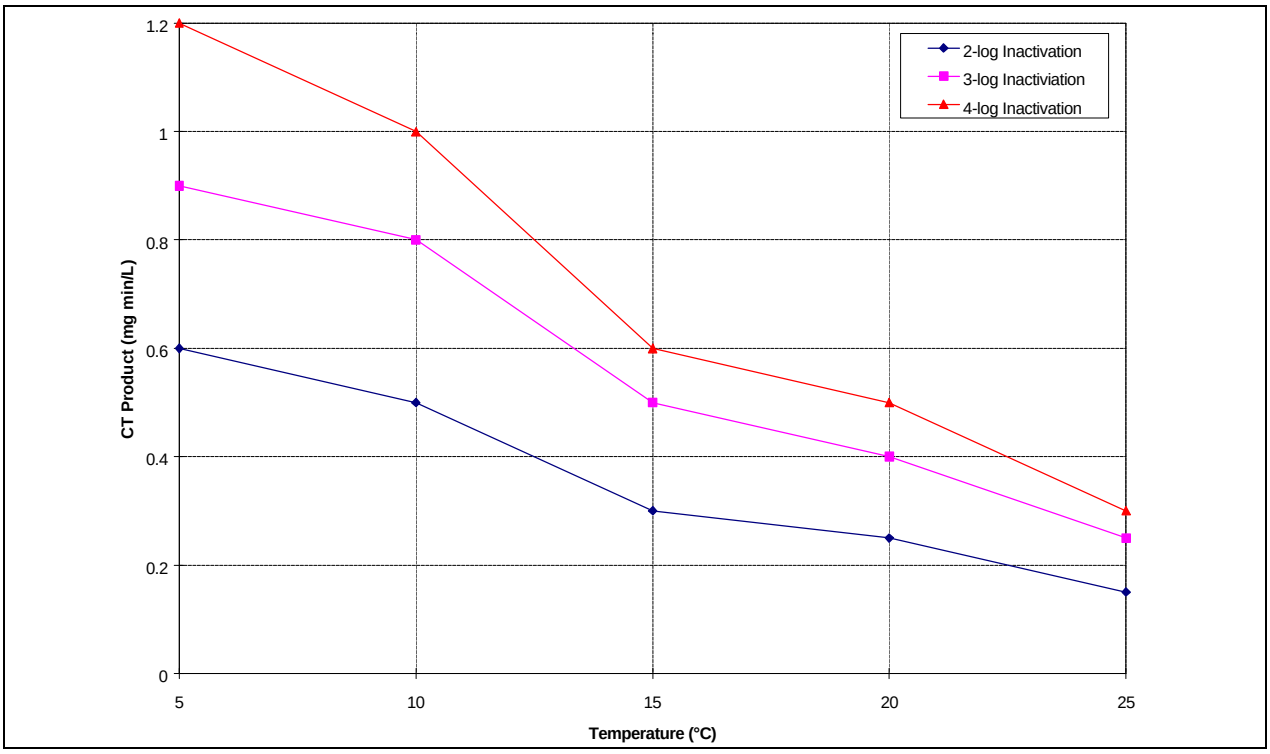



**Figure 3-10. CT Values for Inactivation of Viruses by Ozone (pH 6 to 9)**

**Table 3-8. Summary of Reported Ozonation Requirements for 99 Percent Inactivation of *Cryptosporidium* Oocycts**

| Species | Ozone protocol | Ozone residual (mg/L) | Contact time (min) | Temperature (°C) | CT (mg.min/L) | Reference |
|---|---|---|---|---|---|---|
| *C. baileyi* | Batch liquid, modified batch ozone | 0.6 & 0.8 | 4 | 25 | 2.4 - 3.2 | Langlais et al., 1990 |
| *C. muris* | Flow through contactor, continuous gas | | | 22 - 25 | 7.8 | Owens et al., 1994 |
| *C. parvum* | Batch liquid, batch ozone | 0.50<br>0.50 | 18<br>7.8 | 7<br>22 | 9.0<br>3.9 | Finch et al., 1993 |
| *C. parvum* | Batch liquid, batch ozone | 0.77<br>0.51 | 6<br>8 | Room | 4.6<br>4 | Peeters et al., 1989 |
| *C. parvum* | Batch liquid, continuous gas | 1.0 | 5 & 10 | 25 | 5 - 10 | Korich et al., 1990 |
| *C. parvum* | Flow through contactor, continuous gas | | | 22 - 25 | 5.5 | Owens et al., 1994 |

Ozone dose and contact time (CT) requirements for the inactivation of *Cryptosporidium* oocysts in drinking water when using ozone has not been established similar to the CT values for viruses and *Giardia* cyst inactivation. Inactivation requirements (log removals) for *Cryptosporidium* oocysts have not been established. In addition, as shown in Table 3-8, the CT requirements reported in the literature vary from study to study which adds uncertainty to design CT requirements for specific applications or regulatory needs.

## 3.4 Ozonation Disinfection Byproducts

Ozone does not form halogenated DBPs (TTHMs and HAA5s) when participating in oxidation/reduction reactions with NOM but it does form a variety of organic and inorganic byproducts. Table 3-9 and Figure 3-11 show the principal known byproducts associated with ozonation. However, if bromide ion is present in the raw water halogenated DBPs may be formed. These brominated DBPs appear to pose a greater health risk than non-brominated DBPs.

**Table 3-9. Principal Known Byproducts of Ozonation**

| Disinfectant Byproducts | |
|---|---|
| **Aldehydes** | **Aldo- and Ketoacids** |
| Formaldehyde | Pyruvic acid |
| Acetaldehyde | **Brominated Byproducts*** |
| Glyoxal | Bromate ion |
| Methyl Glyoxal | Bromoform |
| **Acids** | Brominated acetic acids |
| Oxalic acid | Bromopicrin |
| Succinic acid | Brominated acetonitriles |
| Formic acid | **Others** |
| Acetic acid | Hydrogen peroxide |

*Brominated byproducts are produced only in waters containing bromide ion
Source: Singer, 1992.

Although ozone is an effective oxidant and disinfectant, it should not be relied upon as a secondary disinfectant to maintain a residual in the distribution system. Monochloramine is attractive for this purpose because it produces little to no halogenated DBPs. Chlorine is a candidate for secondary disinfectant but the ozonated water may actually produce either more or less DBPs following the addition of free chlorine depending on the nature of the organic material following ozonation unless biologically active filtration precedes the addition of chlorine. The principal benefit of using ozone for controlling DBP formation is that it allows free chlorine to be applied later in the treatment process after precursors have been removed and at lower doses, thereby reducing DBPFP.

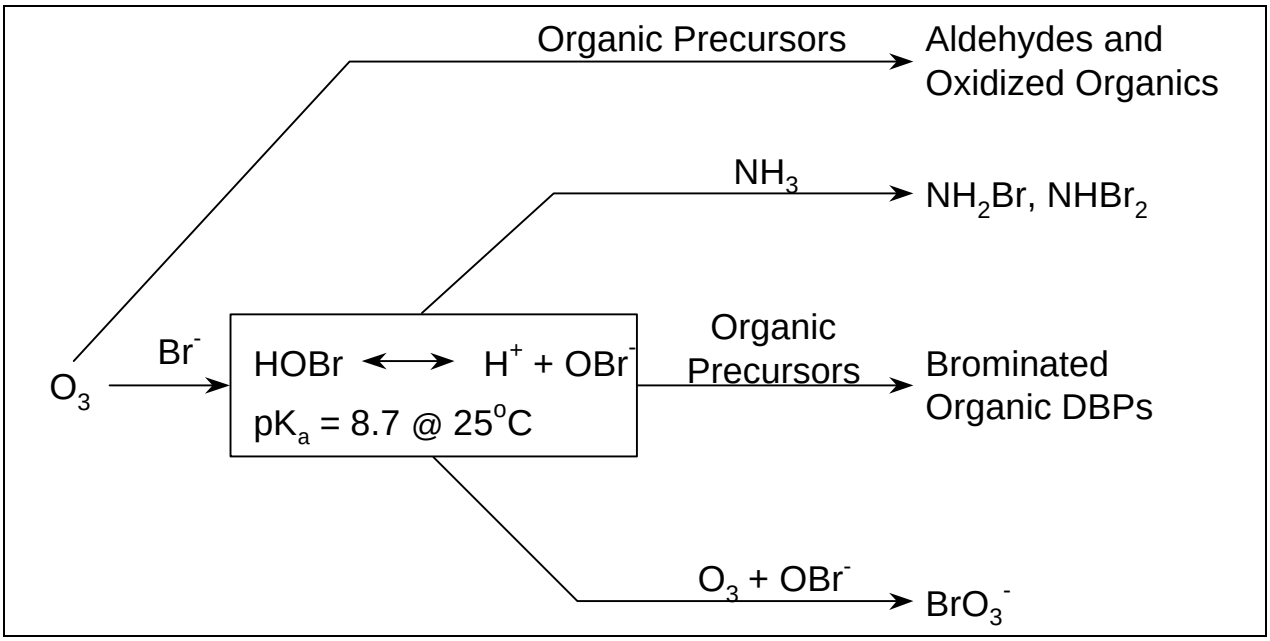


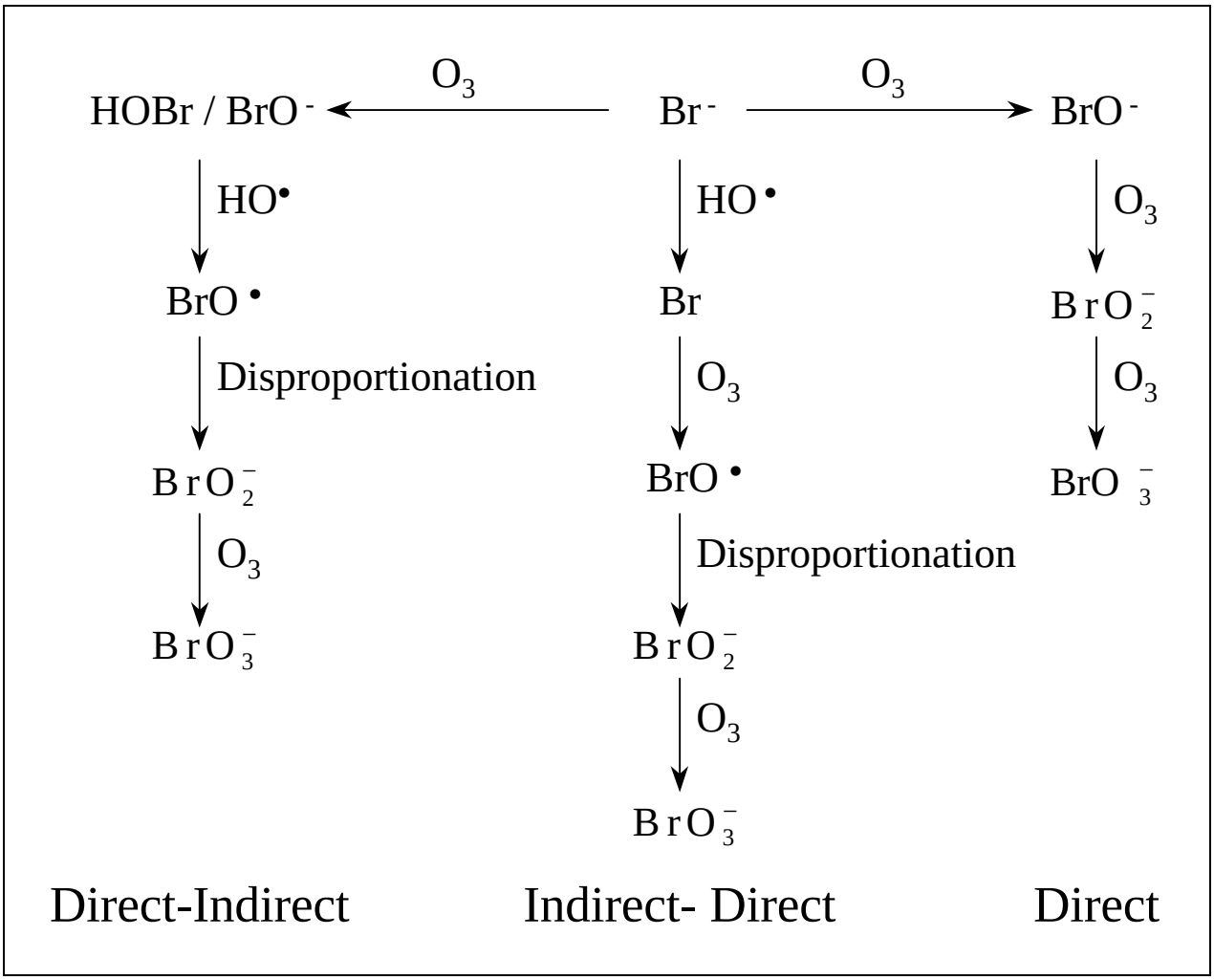

**Figure 3-11. Principal Reactions Producing Ozone Byproducts**

Application of a secondary disinfectant following ozonation requires special consideration for potential interaction between disinfectants. For example, chloral hydrate formation has been

observed when using chlorine as a secondary disinfectant with ozone (McKnight and Reckhow, 1992; Logsdon et al., 1992). The ozonation byproduct of acetaldehyde is a known precursor for chloral hydrate, a byproduct of chlorination. Enhancement of chloral hydrate has not been observed when monochloramine is applied as the secondary disinfectant, or if biologically active filtration is used following ozonation and prior to chlorination (Singer, 1992). Chloropicrin formation from free chlorine appears to be enhanced by pre-ozonation (Hoigné and Bader, 1988) in the absence of biologically active filtration prior to addition of chlorine.

Byproducts such as aldehydes, ketones, acids, and others will be formed upon ozonation of water. The primary aldehydes that have been measured are: formaldehyde, acetaldehyde, glyoxal, and methyl glyoxal (Glaze et al., 1991). Total aldehyde concentration in drinking water disinfected with ozone range from less than 5 μg/L to 300 μg/L, depending on the TOC concentration and the applied ozone to organic carbon ratio (Van Hoof et al., 1985; Yamada and Somiya, 1989; Glaze et al., 1989a; Krasner et al., 1989; Glaze et al., 1991; LeLacheur et al., 1991). Aldehydes with higher molecular weights have also been reported (Glaze et al., 1989b). Other organic byproducts of ozonation are indicated in Table 3-9

Ozonation of a source water containing bromide ion can produce brominated byproducts, the brominated analogues of the chlorinated DBPs. Song et al. (1997) found that bromate ion formation is an important consideration for waters containing more than 0.10 mg/L bromide ion. These brominated byproducts include bromate ion, bromoform, the brominated acetic acids and acetonitriles, bromopicrin, and cyanogen bromide (if ammonia is present). An ozone dose of 2 mg/L produced 53 μg/L of bromoform and 17 μg/L of dibromoacetic acid in a water containing 2 mg/L of bromide ion (McGuire et al., 1990). Ozonation of the same water spiked with 2 mg/L bromide ion showed cyanogen bromide formation of 10 μg/L (McGuire et al., 1990). Furthermore, ozone may react with the hypobromite ion to form bromate ion (Amy and Siddiqui, 1991; Krasner et al., 1993), a probable human carcinogen (Regli et al., 1992). Bromate ion concentrations in ozonated water up to 60 μg/L have been reported (Amy and Siddiqui, 1991; Krasner et al., 1993). Note that the amount of bromide ion incorporated into the measured DBPs accounts for only one-third of the total raw water bromide ion concentration. This indicates that other brominated DBPs exist that are not yet identified (Krasner et al., 1989; MWDSC and JMM, 1992). Figure 3-12 shows the major pathways for bromate ion formation.




Source: Song et al., 1997.

**Figure 3-12. Main Pathways of Bromate Ion Formation when Ozone Reacts with Bromide Ion**

## 3.4.1 Ozone Byproduct Control

The primary factors affecting the speciation and concentrations of brominated byproducts are pH and the ozone-to-bromide ion and TOC-to-bromide ion ratios (Singer, 1992). Bromate ion formation can be controlled by ozonation at acidic pH values of which hypobromous acid dominates over the now absent hypobromite ion (Haag and Hoigné, 1984; Amy and Siddiqui, 1991; Krasner et al., 1993). Conversely, under alkaline pH conditions, ozone can oxidize the hypobromous acid further to produce bromate ion. At low pH values, the brominated organic byproducts are favored, while at greater pH values, bromate ion formation is favored. Therefore, the application of ozone may be limited for source waters containing bromide ion. Bromate ion formation can be controlled by lowering the ambient bromide ion concentration, lowering the ozone residual, and lowering the ozonation pH. The addition of ammonia with ozonation to form bromamines reduces the formation of both bromate ion and organic byproducts (Amy and Siddiqui, 1991; MWDSC and JMM, 1992). However, ammonia can act as a nutrient for nitrifying bacteria.

The organic acid and aldehyde byproducts of ozonation discussed above appear to be readily biodegradable and are a component of the assimilable organic carbon (AOC) or biodegradable organic carbon (BDOC). Ozonation increases the BOM by oxidation. Therefore, if water disinfected with ozone is coupled with a biologically active process (i.e., biological active carbon), removal of these biodegradable byproducts can be reduced. The use of biologically active filters, maintained by discontinuing the application of a disinfectant to the filters, has been shown to successfully remove aldehydes and other compounds representing a portion of the BDOC in a water (Bablon et al., 1988; Rittman, 1990; Reckhow et al., 1992). See Section 3.3.4 for a detailed discussion on biological active filters.

A recent study has shown that bromate ion and brominated organics can be controlled during ozonation by the following techniques (Song et al., 1997):

- Low pH decreases bromate ion formation while increasing brominated organic formation;
- Ammonia addition with short ozone contact time decreases both bromate ion and brominated organic formation;
- Hydrogen peroxide decreases brominated organic formation and may increase or decrease bromate ion formation, depending on other water quality parameters; and
- Low ozone DOC ratio leads to low bromate ion and brominated organic formation.

## 3.5 Status of Analytical Methods

During operation of an ozonation system it is necessary to analyze for ozone in both the liquid and gas phase to determine the applied ozone dose, ozone transfer efficiency and (for primary disinfection) residual ozone level. The gas stream exiting the ozone generator is monitored for ozone content to determine the applied ozone dose. The off-gas exiting the ozone contactor is monitored to determine the amount of ozone transferred to the liquid phase in the contactor and to calculate the ozone transfer efficiency. The disinfected water exiting the ozone contactor is monitored for residual ozone to ensure that CT values are met.

In addition, the ambient air in any ozone generating or handling room and ozone destruct off-gas are monitored for ozone concentration to protect workers in the event of leakages or destruct system failure.

### 3.5.1 Monitoring of Gas Phase Ozone

Points in an ozone treatment system where gas-phase ozone is monitored include:

- Ozone generator output;
- Contactor off-gas;
- Ozone destruct off-gas; and

- Ambient air in ozone process areas.

The range of ozone concentrations to be measured in the gas phase varies from less than 0.1 ppm by volume (0.2 mg/$m^3$ NTP) in ambient air and ozone destruct off-gas, to 1 to 2 percent (10 g/$m^3$ NTP) in contactor off-gas, to as high as 15 percent by weight (200 g/$m^3$ NTP) in the ozone generator output.

Analytical methods for monitoring gas-phase ozone include:

- UV absorption;
- Iodometric methods;
- Chemiluminescence; and
- Gas-Phase titration.

Table 3-10 presents the working range, expected accuracy and precision, operator skill level required, interferences, and current status for gas phase ozone analysis.

#### 3.5.1.1 UV Absorption

Gaseous ozone absorbs light in the short UV wavelength region with a maximum absorbance at 253.7 nm (Gordon et al., 1992). Instruments for measuring ozone by the absorption of UV radiation are supplied by several manufacturers for gas concentrations below 0.5 ppm by volume (1 g/$m^3$ NTP). In general, these instruments measure the amount of light absorptions when no ozone is present and the amount of light absorptions when ozone is present. The meter output is the difference of the two readings, which is directly related to the actual amount of ozone present. The International Ozone Association (IOA) has accepted this procedure (IOA, 1989).

## Table 3-10. Characteristics and Comparisons of Gas-Phase Ozone Analytical Methods

| Type of Test | Working Range (mg/L) | Expected Accuracy (± %) | Expected Precision (± %) | Skill Level[a] | Interferences | pH Range | Field Test | Automated Test | Current Status |
|---|---|---|---|---|---|---|---|---|---|
| UV Absorption | 0.5 - 50,000 | 2 | 2.5 | 1/2 | None | NA | Yes | Yes | Recommended |
| Stripping Absorption Iodometry | 0.5 - 100 | 1 - 35 | 1 - 2 | 2 | $SO_2$, $NO_2$ | NA | Yes | No | Not Recommended |
| Chemiluminescence | 0.005 - 1 | 7 | 5 | 1/2 | None | NA | Yes | Yes | Recommended |
| Gas-Phase Titration | 0.005 - 30 | 8 | 8.5 | 2 | None | NA | Yes | No | Not Recommended |

Source: Gordon et al., 1992.
Notes: [a] Operator Skill Levels: 1 = minimal, 2 = good technician, 3 = experienced chemist. NA = Not Applicable

### 3.5.1.2 Iodometric Methods

Iodometric procedures have been used for all of the ozone concentration ranges encountered in water treatment plants (Gordon et al., 1992). This includes measurement of ozone directly from the generator and of ozone as stripped from aqueous solution. For the iodometric method, the ozone-containing gas is passed into an aqueous solution containing excess potassium iodide.

All other oxidizing materials act as interferences with iodometry (Gordon et al., 1992). Nitrogen oxides (that may be present when ozone is generated in air) also act as interferences to iodometric methods. The effects of nitrogen oxides may be eliminated by passing the ozone-containing gas through absorbents such as potassium permanganate that are specific for nitrogen oxide gases. However, no iodometric method is recommended for the determination of ozone in solution because of the unreliability of the method (Gordon et al., 1989).

### 3.5.1.3 Chemiluminescence

Chemiluminescence methods can be used for the determination of low concentrations of ozone in the gas phase (Gordon et al., 1992). One of the most commonly used methods is ethylene chemiluminescence. Gas-phase ozone can be measured using the chemiluminescent reaction between ethylene and ozone. This method is specific to ozone and is suitable for measurement of ozone in the ambient air. The ethylene chemiluminescence procedure was adopted in 1985 by the EPA as its reference method for determining ozone in the ambient atmosphere (McKee et al., 1975). Chemiluminescent instruments are approved by the EPA for monitoring ambient ozone concentrations of 0 to 0.5 or 0 to 1.0 ppm by volume. With regular calibration, this type of instrument is capable of providing reliable analysis of any ozone in the ambient air from an ozonation plant.

An alternative to ethylene chemiluminescence is rhodamine B/gallic acid chemiluminescence, which avoids the handling of ethylene (Gordon et al., 1992). This alternative method is considerably more complex than the more common ethylene chemiluminescence instruments. The sensitivity of this method tends to drift and a procedure has been developed by which corrections are made for the sensitivity on a frequent basis (Van Dijk and Falkenberg, 1977). Given the wide availability of ethylene chemiluminescence monitors and their approval by EPA, ethylene monitors should be considered before rhodamine B/gallic acid monitors.

### 3.5.1.4 Gas-Phase Titration

Two gas-phase titration methods have been studied as possible calibration methods for ambient ozone analyzers and monitors (Gordon et al., 1992). These procedures are based on titration with nitric oxide and back titration of excess nitric oxide (Rehme et al., 1980). These gas phase titration procedures, evaluated by EPA, were compared with UV absorption and iodometry as calibration methods for ethylene chemiluminescent ambient air ozone analyzers. As a result of these comparisons, UV absorption has been specified as the method of calibration for ambient ozone

analyzers. Therefore, gas-phase titration methods are not recommended for use at ozonation facilities (Gordon et al., 1992).

## 3.5.2 Monitoring of Liquid Phase Residual Ozone

There are numerous methods for monitoring ozone in aqueous solutions. Gordon et al. (1992) recommended the following methods for analyzing residual ozone:

- Indigo colorimetric method;
- Acid chrome violet K (ACVK) method;
- Bis-(Terpyridine) iron(II) method; and
- Stripping into the gas-phase.

Table 3-11 shows the working range, expected accuracy and precision, operator skill level required, interferences and current status for liquid phase residual ozone analytical methods.

### *3.5.2.1 Indigo Colorimetric Method*

The indigo colorimetric method is the only method for monitoring residual ozone in *Standard Methods*, 1995. The indigo colorimetric method is sensitive, precise, fast, and more selective for ozone than other methods. There are two indigo colorimetric methods: spectrophotometric and visual. For the spectrophotometric procedure the lower limit of detection is 2 μg/L, while for the visual procedure the detection limit is 10 μg/L.

Hydrogen peroxide, chlorine, manganese ions, ozone decomposition products, and the products of organic ozonation exhibit less interference with the indigo colorimetric method than any of the other methods (Langlais et al., 1991). However, the masking of chlorine in the presence of ozone can make the indigo method problematic. In the presence of hypobromous acid, which forms during ozonation of bromide-ion containing methods, an accurate measurement cannot be made with this method (Standard Methods, 1995).

## Table 3-11. Characteristics and Comparisons of Residual Ozone Analytical Methods

| Type of Test | Working Range (mg/L) | Expected Accuracy (± %) | Expected Precision (± %) | Skill Level[a] | Interferences | pH Range | Field Test | Automated Test | Current Status |
|---|---|---|---|---|---|---|---|---|---|
| Indigo Colorimetric, Spectrophotometric | 0.01 - > 0.3 | 1 | 0.5 | 1 | Chlorine, Manganese ions, Bromine, Iodine | 2 | No | Yes | Recommended |
| Indigo Colorimetric, Visual | 0.01 - > 0.1 | 1 | 0.5 | 1 | Chlorine, Manganese ions, Bromine, Iodine | 2 | No | Yes | Recommended |
| ACVK | 0.05 - 1 | NR | NR | 1 | Manganese ions > 1 mg/L, Chlorine > 10 mg/L | 2 | No | No | Recommended |
| Bis-(Terpyridine) Iron(II) | 0.05 - 20 | 2.7 | 2.1 | 3 | Chlorine | < 7 | No | Yes | Recommended Lab Test |

Source: Gordon et al., 1992.
Notes: [a] Operator Skill Levels: 1 = minimal, 2 = good technician, 3 = experienced chemist. NR = Not reported in literature cited by referenced source.

#### 3.5.2.2 ACVK Method

ACVK is readily bleached by ozone, which serves as the basis for this spectrophotometric procedure first reported by Masschelein (1977). The procedure has been developed further for the measurement of ozone in the presence of chlorine (Ward and Larder, 1973).

The advantages of this method (Langlais et al., 1991) are:

- Its ease of execution and stability of the dye reagent solution;
- A linear calibration curve over the range of 0.05 to 1.0 mg/L of ozone; and
- The apparent lack of interferences from low levels of manganese, chlorine or combined chlorine (up to 10 mg/L), organic peroxides, and other organic oxidation products.

#### 3.5.2.3 Bis-(Terpyridine)Iron(II) Method

Bis-(Terpyridine)Iron(II) in dilute hydrochloric acid solution reacts with ozone to change the absorbance spectra measured at 552 nm (Tomiyasu and Gordon, 1984). The only known interferent is chlorine. However, chlorine interferences can be masked by the addition of malonic acid. Alternatively, since the reaction between chlorine and the reagent is slower than the reaction with ozone, the spectrophotometric measurements can be carried out immediately after reagent mixing to reduce the chlorine interference. Chlorine dioxide does not interfere (Gordon et al, 1992).

The primary advantages of the Bis-(Terpyridine)Iron(II) Method are its lack of interferences, low limits of detection (4 μg/L), broad working range (up to 20 mg/L), excellent reproducibility, and agreement with the indigo colorimetric method.

#### 3.5.2.4 Stripping into the Gas-Phase

In this indirect procedure, residual ozone is stripped from the solution using an inert gas. The amount of ozone present in the gas phase is then analyzed by gas-phase analytical methods such as UV absorption or chemiluminescence described previously. This stripping technique was developed to minimize the complications caused by the presence of other oxidants in solution.

The success of this procedure initially depends upon the ability to strip ozone from the treated water without any decomposition. Since stripping conditions such as temperature, pH, and salinity can vary, the reliability of this method is suspect (Langlais et al., 1991).

### 3.5.3 Bromate Monitoring for Systems Using Ozone

The DBPR requires that community water systems and non-transient non-community water systems that use ozone for disinfection or oxidation must monitor their system for bromate. These systems are required to take one sample per month for each treatment plant with samples collected at the entrance to the distribution system while the ozonation system is operating under normal conditions.

The DBPR provides reduced monitoring opportunities (i.e., quarterly rather than monthly samples) if the system demonstrates that the average source water bromide concentration is less than 0.05 mg/L based upon representative monthly bromide measurements for one year. Systems can remain on the reduced monitoring schedule until the running annual average source water bromide concentration, computed quarterly, is equal to or greater than 0.05 mg/L based upon representative monthly measurements.

For compliance monitoring for bromate, systems must use the ion chromatography analytical method as specified in *USEPA Method 300.1, Determination of Inorganic Anions in Drinking Water by Ion Chromatography, Revision 1.0* (USEPA, 1997).

If the average of samples covering any consecutive four-quarter period exceeds the MCL, the system is in violation of the MCL and must notify the public pursuant to 40 CFR §141.32. The system must also report to the State pursuant to 40 CFR §141.134. If the system fails to complete 12 consecutive months' monitoring, compliance with the MCL for the last four-quarter compliance period must be based on an average of the available data.

# 3.6 Operational Considerations

## 3.6.1 Process Considerations

Because ozone is such a strong oxidant, it will react with many organic and inorganic compounds present in the water. Ozone is used to remove tastes and odors by breaking down organic compounds, and to aid in the removal of iron and manganese by oxidizing these compounds to less soluble forms. These demands should be satisfied before any ozone is available to satisfy primary disinfection requirements. The presence and concentration of these compounds can dictate the location of ozone addition, depending on the process goals.

Stolarik and Christie (1997) present the results of 10 years of operation at the 600 mgd Los Angeles Aqueduct Filtration and Ozone Facility. Operational experiences at this facility showed lower particle counts (greater than one micron) with ozone use. The optimum ozone concentration in the gas phase applied was found to be 6 percent when using the cryogenic oxygen production facilities, and 4 to 5 percent when using liquid oxygen (LOX).

## 3.6.2 Space Requirements

Storage of LOX is subject to regulations in building and fire codes. These regulations will impact the space requirements and may dictate the construction materials of adjacent structures if the certain setback requirements cannot be met. In general, the footprint for ozone generated from air is smaller than that required for chloramination and chloride dioxide applications. However, the footprint area for ozone generated from pure oxygen is comparable to that of chlorine dioxide because of the additional area needed for storage.

## 3.6.3 Material Selection

Ozone-resistant materials should be used from the ozone generators through the off-gas destruct unit. If oxygen is used for the feed gas, oxygen resistant materials should be used up to the generators. Pure oxygen piping should be specially cleaned after installation for oxygen service, which increases construction cost. Materials for air preparation systems can be those normally used for compressed air systems. Langlais et al. (1991) recommended that piping beyond the desiccant dryers be ozone-resistant, as some backflow and ozone diffusion can occur. If a receiver is provided following the desiccant dryer, the piping should be ozone-resistant, downstream of the pressure regulator. Ozone-resistant (oxygen resistant as well if high purity oxygen is the feed gas) check valves should be placed in the piping ahead of the generator.

Ozone-resistant materials include the austenitic (300 series) stainless steels, glass and other ceramics, Teflon and Hypalon, and concrete. The 304 series stainless steels can be used for "dry" ozone gas (also for oxygen), 316 series should be used for "wet" service. Wet service includes piping in the contactors and all off-gas piping and the off-gas destruct unit. Teflon or Hypalon should be used for gasket materials. Concrete should be manufactured from Type II or Type IV cement. Typical practice in the United States is to provide 3 inches of cover for reinforcing to prevent corrosion by either ozone gas or ozone in solution, although Fonlupt (1979) reports that 4 cm (1.13 inches) is adequate for protection. Hatches for access into contactors should be fabricated from 316 series stainless steels and provided with ozone-resistant seals.

## 3.6.4 Ozone System Maintenance

Stolarik and Christie (1997) provide a good overview of the operational and maintenance requirements during the 10 years of operating the 600 mgd Los Angeles Aqueduct Filtration and Ozone Plant. The ozone system has been available 97.1 percent of the time over the 10 year period.

Fuse failure and generator cleaning comprised the major maintenance chores on the ozone generators during the first years. Fuse failure was caused by a malfunction when its glass dielectric tube failed. Vessels are cleaned every three years or when exit gas temperatures rise due to $Fe_3O_4$ deposits on the ground electrode/heat exchanger surfaces.

Rod shaped ceramic diffusers worked well as ozone diffusers for the initial two years. These were replaced by sintered stainless steel and ultimately a modified ceramic diffuser.

## 3.6.5 Ozone Safety

Concern for safety even at the risk of being overcautious, would be to follow practices that have been successfully applied to other oxidants over the years. This would be to generally isolate the ozonation system from the remainder of the plant. This should not be interpreted to mean a separate building, but rather separate rooms, separate exterior entrances, separate heating and ventilation systems, noise control, etc. This method already is manifested in some of the European ozonation plants, but on a lesser scale.

Ozone generators should be housed indoors for protection from the environment and to protect personnel from leaking ozone in the case of a malfunction. Ventilation should be provided to prevent excess temperature rise in the generator room, and to exhaust the room in the case of a leak. Adequate space should be provided to remove the tubes from the generator shell and to service the generator power supplies. Air prep systems tend to be noisy; therefore, it is desirable to separate them from the ozone generators. Off-gas destruct units can be located outside if the climate is not too extreme. If placed inside, an ambient ozone detector should be provided in the enclosure. All rooms should be properly ventilated, heated, and cooled to match the equipment-operating environment.

Continuous monitoring instruments should be maintained to monitor levels of ozone in the rooms. Self-contained breathing apparatuses should be located in hallways outside the rooms liable to ozone hazards. Ambient ozone exposure levels, which have been proposed by appropriate U.S. organizations, are summarized below. The maximum recommended ozone levels are as follows:

- **Occupational Safety and Health Administration.** The maximum permissible exposure to airborne concentrations of ozone not in excess of 0.1 mg/L (by volume) averaged over an eight-hour work shift.

- **American National Standards Institute/American Society for testing Materials (ANSI/ASTM).** Control occupational exposure such that the worker will not be exposed to ozone concentrations in excess of a time weighted average of 0.1 mg/L (by volume) for eight hours or more per workday, and that no worker be exposed to a ceiling concentration of ozone in excess of 0.3 mg/L 9by volume) for more than ten minutes.

- **American Conference of Government Industrial Hygienists (ACGIH).** Maximum ozone level of 0.1 mg/L (by volume) for a normal eight hour work day or 40 hour work week, and a maximum concentration of 0.3 mg/L (by volume) for exposure of up to 15 minutes.

- **American Industrial Hygiene Association.** Maximum, concentration for eight hour exposure of 0.1 mg/L (by volume).

There is a question of whether prolonged exposure to ozone may impair a worker's ability to smell or be aware of ozone levels at less than critical levels. Awareness of an odor of ozone should not be relied upon. Instrumentation and equipment should be provided to measure ambient ozone levels and perform the following safety functions:

- Initiate an alarm signal at an ambient ozone level of 0.1 mg/L (by volume). Alarms should include warning lights in the main control panel and at entrances to the ozonation facilities as well as audible alarms.

- Initiate a second alarm signal at ambient ozone levels of 0.3 mg/L (by volume). This signal would immediately shut down ozone generation equipment and would initiate a second set of visual and audible alarms at the control panel and at the ozone generation facility entrances. An

emergency ventilation system capable of exhausting the room within a period of 2 to 3 minutes also would be interconnected to the 0.3 mg/L ozone level alarm.

Ozone gas is a hazardous gas and should be handled accordingly. Ambient ozone levels should be monitored and equipment shut- down and alarmed when levels exceed 0.1 ppm. Emergency ventilation is typically provided for enclosed areas. Building and fire codes will provide additional guidance. The OSHA exposure limit for an 8-hour shift is 0.1 ppm by volume. The pungent odor of ozone will provide warning to operators of any possible ozone leak.

# 3.7 Summary

## 3.7.1 Advantages and Disadvantages of Ozone Use

The following list highlights selected advantages and disadvantages of using ozone as a disinfection method for drinking water (Masschelein, 1992). Because of the wide variation of system size, water quality, and dosages applied, some of these advantages and disadvantages may not apply to a particular system.

### Advantages

- Ozone is more effective than chlorine, chloramines, and chlorine dioxide for inactivation of viruses, *Cryptosporidium*, and *Giardia*.
- Ozone oxidizes iron, manganese, and sulfides.
- Ozone can sometimes enhance the clarification process and turbidity removal.
- Ozone controls color, taste, and odors.
- One of the most efficient chemical disinfectants, ozone requires a very short contact time.
- In the absence of bromide, halogen-substitutes DBPs are not formed.
- Upon decomposition, the only residual is dissolved oxygen.
- Biocidal activity is not influenced by pH.

### Disadvantages

- DBPs are formed, particularly by bromate and bromine-substituted DBPs, in the presence of bromide, aldehydes, ketones, etc.
- The initial cost of ozonation equipment is high.
- The generation of ozone requires high energy and should be generated on-site.
- Ozone is highly corrosive and toxic.
- Biologically activated filters are needed for removing assimilable organic carbon and biodegradable DBPs.
- Ozone decays rapidly at high pH and warm temperatures.
- Ozone provides no residual.

- Ozone requires higher level of maintenance and operator skill.

## 3.7.2 Summary Table

Table 3-12 presents a summary of the considerations for the use of ozone as a disinfectant.

**Table 3-12. Summary of Ozone Disinfection Considerations**

| Consideration | Description |
|---|---|
| Generation | Because of its instability, ozone should be generated at the point of use. Ozone can be generated from oxygen present in air or high purity oxygen. The feed gas source should be clean and dry, with a maximum dewpoint of -60°C. Ozone generation consumes power at a rate of 8 to 17 kWhr/kg $O_3$. Onsite generation saves a lot of storage space. |
| Primary uses | Primary uses include primary disinfection and chemical oxidation. As an oxidizing agent, ozone can be used to increase the biodegradability of organic compounds destroys taste and odor control, and reduce levels of chlorination DBP precursors. Ozone should not be used for secondary disinfection because it is highly reactive and does not maintain an appreciable residual level for the length of time desired in the distribution system. |
| Inactivation efficiency | Ozone is one of the most potent and effective germicide used in water treatment. It is effective against bacteria, viruses, and protozoan cysts. Inactivation efficiency for bacteria and viruses is not affected by pH; at pH levels between 6 and 9. As water temperature increases, ozone disinfection efficiency increases. |
| Byproduct formation | Ozone itself does not form halogenated DBPs; however, if bromide ion is present in the raw water or if chlorine is added as a secondary disinfectant, halogenated DBPs, including bromate ion may be formed. Other ozonation byproducts include organic acids and aldehydes. |
| Limitations | Ozone generation is a relatively complex process. Storage of LOX (if oxygen is to be the feed gas) is subject to building and fire codes. |
| Points of application | For primary disinfection, ozone addition should be prior to biofiltration/filtration and after sedimentation. For oxidation, ozone addition can be prior to coagulation/sedimentation or filtration depending on the constituents to be oxidized. |
| Safety considerations | Ozone is a toxic gas and the ozone production and application facilities should be designed to generate, apply, and control this gas, so as to protect plant personnel. Ambient ozone levels in plant facilities should be monitored continuously. |

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# 4. CHLORINE DIOXIDE

Since the beginning of the twentieth century, when it was first used at a spa in Ostend, Belgium, chlorine dioxide has been known as a powerful disinfectant of water. During the 1950s, it was introduced more generally as a drinking water disinfectant since it provided less organoleptic hindering than chlorine. Approximately 700 to 900 public water systems use chlorine dioxide to treat potable water (Hoehn, 1992). Today, the major uses of chlorine dioxide are:

- CT disinfection credit;
- Preoxidant to control tastes and odor;
- Control of iron and manganese; and
- Control of hydrogen sulfide and phenolic compounds.

## 4.1 Chlorine Dioxide Chemistry

### 4.1.1 Oxidation Potential

The metabolism of microorganisms and consequently their ability to survive and propagate are influenced by the oxidation reduction potential (ORP) of the medium in which it lives (USEPA, 1996).

Chlorine dioxide ($ClO_2$) is a neutral compound of chlorine in the +IV oxidation state. It disinfects by oxidation; however, it does not chlorinate. It is a relatively small, volatile, and highly energetic molecule, and a free radical even while in dilute aqueous solutions. At high concentrations, it reacts violently with reducing agents. However, it is stable in dilute solution in a closed container in the absence of light (AWWA, 1990). Chlorine dioxide functions as a highly selective oxidant due to its unique, one-electron transfer mechanism where it is reduced to chlorite ($ClO_2^-$) (Hoehn et al., 1996). The pKa for the chlorite ion, chlorous acid equilibrium, is extremely low at pH 1.8. This is remarkably different from the hypochlorous acid/hypochlorite base ion pair equilibrium found near neutrality, and indicates the chlorite ion will exist as the dominant species in drinking water. The oxidation reduction of some key reactions are (CRC, 1990):

$$ClO_{2(aq)} + e^- = ClO_2^- \qquad E° = 0.954V$$

Other important half reactions are:

$$ClO_2^- + 2H_2O + 4e^- = Cl^- + 4OH^- \qquad E° = 0.76V$$

$$ClO_3^- + H_2O + 2e^- = ClO_2^- + 2OH^- \qquad E° = 0.33V$$

$$ClO_3^- + 2H^+ + e^- = ClO_2 + H_2O \qquad E° = 1.152V$$

In drinking water, chlorite ($ClO_2^-$) is the predominant reaction endproduct, with approximately 50 to 70 percent of the chlorine dioxide converted to chlorite and 30 percent to chlorate ($ClO_3^-$) and chloride ($Cl^-$) (Werdehoff and Singer, 1987).

# 4.2 Generation

## 4.2.1 Introduction

One of the most important physical properties of chlorine dioxide is its high solubility in water, particularly in chilled water. In contrast to the hydrolysis of chlorine gas in water, chlorine dioxide in water does not hydrolyze to any appreciable extent but remains in solution as a dissolved gas (Aieta and Berg, 1986). It is approximately 10 times more soluble than chlorine (above 11°C), while it is extremely volatile and can be easily removed from dilute aqueous solutions with minimal aeration or recarbonation with carbon dioxide (e.g. softening plants). Above 11 to 12°C, the free radical is found in gaseous form. This characteristic may affect chlorine dioxide's effectiveness when batching solutions and plumbing appropriate injection points. Other concerns are the increased difficulty in analyzing for specific compounds in the presence of many interfering compounds/residual longevity and volatility of gaseous compounds. In the gaseous form, the free radicals also react slowly with water. The reaction rate is 7 to 10 million times slower than that of the hydrolysis rate for chlorine gas (Gates, 1989).

Chlorine dioxide cannot be compressed or stored commercially as a gas because it is explosive under pressure. Therefore, it is never shipped. Chlorine dioxide is considered explosive at higher concentrations which exceed 10 percent by volume in air, and its ignition temperature is about 130°C (266°F) at partial pressures (National Safety Council Data Sheet 525 – $ClO_2$, 1967). Strong aqueous solutions of chlorine dioxide will release gaseous chlorine dioxide into a closed atmosphere above the solution at levels that may exceed critical concentrations. Some newer generators produce a continuous supply of dilute gaseous chlorine dioxide in the range of 100 to 300 mm-Hg (abs) rather than in an aqueous solution (National Safety Council, 1997). For potable water treatment processes, aqueous solutions between 0.1 and 0.5 percent are common from a number of current generation technologies.

Most commercial generators use sodium chlorite ($NaClO_2$) as the common precursor feedstock chemical to generate chlorine dioxide for drinking water application. Recently, production of chlorine dioxide from sodium chlorate ($NaClO_3$) has been introduced as a generation method where in $NaClO_3$ is reduced by a mixture of concentrated hydrogen peroxide ($H_2O_2$) and concentrated sulfuric acid ($H_2SO_4$). Chlorate-based systems have traditionally been used in pulp and paper applications, but have recently been tested full-scale at two U.S. municipal water treatment plants. This is an emerging technology in the drinking water field and is not discussed in this guidance manual.

## 4.2.2 Chlorine Dioxide Purity

Chlorine dioxide generators are operated to obtain the maximum production (yield) of chlorine dioxide, while minimizing free chlorine or other residual oxidant formation. The specified yield for chlorine dioxide generators is typically greater than 95 percent. In addition, the measurable excess chlorine should be less than 2 percent by weight in the generator effluent. Generator yield is defined as (Gordon et al., 1990):

$$Yield = \frac{[ClO_2]}{[ClO_2]+[ClO_2^-]+\left(\frac{67.45}{83.45}\right)[ClO_3^-]} \times 100$$

Where: $[ClO_2]$ = Chlorine dioxide concentration, mg/L.

$[ClO_2^-]$ = Chlorite concentration, mg/L.

$[ClO_3^-]$ = Chlorate concentration, mg/L.

$\left(\frac{67.45}{83.45}\right)$ = Molecular weight ratio of $ClO_2^-$ to $ClO_3^-$.

Since any chlorite ion fed to the generator may result in the formation of $ClO_2$, $ClO_2^-$, or $ClO_3^-$, the purity of the resultant mixture can be calculated using the concentrations of each of the species from appropriate analytical measurements. The determination of purity requires neither flow measurement, mass recoveries, nor manufacturer-based methods to determine production "yield," "theoretical yield," "efficiency," or conversion for any precursor feedstock. This approach does not require flow measurements that can introduce up to 5 percent error in the calculations.

Utilities that use chlorine dioxide should measure excess chlorine (as FAC) in the generator effluent in addition to the $ClO_2^-$ related species. FAC may appear as false $ClO_2$ residuals for CT purposes, or result in the formation of chlorinated DBPs if high, relative to the $ClO_2$ level in the generated mixture. Excess chlorine is defined as:

$$\text{Excess } Cl_2 = \frac{[Cl_2]}{[ClO_2]+[ClO_2^-]+\left(\frac{67.45}{83.45}\right)[ClO_3^-]\times\frac{70.91}{2\times 67.45}} \times 100$$

Where: $\frac{70.91}{(2\times 67.45)}$ = stoichiometric and molecular weight ratio of $Cl_2$ to $ClO_2^-$.

The following represents a summarily simpler equation that substantially resolves the problems of different equipment-specific calibration methods, chlorine-contaminated $ClO_2$, or low efficiency conversion of either chlorite- or chlorate-based precursor material.

$$\text{Purity} = \frac{[ClO_2]}{[ClO_2]+[FAC]+[ClO_2^-]+[ClO_3^-]} \times 100$$

This practical (weight-based) calculation permits a variety of approved analytical methods (discussed in section 4.6) to be used to assess generator performance on unbiased scientific principles, rather than non-standardized manufacturer specifications.

## 4.2.3 Methods of Generating Chlorine Dioxide

For potable water application, chlorine dioxide is generated from sodium chlorite solutions. The principal generation reactions that occur in the majority of generators have been known for a long time. Chlorine dioxide can be formed by sodium chlorite reacting with gaseous chlorine ($Cl_{2(g)}$), hypochlorous acid (HOCl), or hydrochloric acid (HCl). The reactions are:

$$2NaClO_2 + Cl_{2(g)} = 2ClO_{2(g)} + 2NaCl \quad [1a]$$

$$2NaClO_2 + HOCl = 2ClO_{2(g)} + NaCl + NaOH \quad [1b]$$

$$5NaClO_2 + 4HCl = 4ClO_{2(g)} + 5NaCl + 2H_2O \quad [1c]$$

Reactions [1a], [1b], and [1c] explain how generators can differ even though the same feedstock chemicals are used, and why some should be pH controlled and others are not so dependent on low pH. In most commercial generators, there may be more than one reaction taking place. For example, the formation and action of hypochlorous acid as an intermediate (formed in aqueous solutions of chlorine) often obscures the "overall" reaction for chlorine dioxide production.

Table 4-1 provides information on some types of available commercial generators. Conventional systems react sodium chlorite with either acid, aqueous chlorine, or gaseous chlorine. Emergent technologies identified in Table 4-1 include electrochemical systems, a solid chlorite inert matrix (flow-through gaseous chlorine) and a chlorate-based emerging technology that uses concentrated hydrogen peroxide and sulfuric acid.

## Table 4-1. Commercial Chlorine Dioxide Generators

| GENERATOR TYPE | MAIN REACTIONS Reactants, byproducts, key reactions, and chemistry notes | SPECIAL ATTRIBUTES |
|---|---|---|
| ACID-CHLORITE:<br>(Direct Acid System) | $4HCl + 5NaClO_2 \rightarrow 4ClO_{2(aq)} + ClO_3^-$<br>• Low pH<br>• $ClO_3^-$ possible<br>• Slow reaction rates | Chemical feed pump interlocks required.<br>Production limit ~ 25-30 lb/day.<br>Maximum yield at ~80% efficiency. |
| AQUEOUS CHLORINE-CHLORITE:<br>($Cl_2$ gas ejectors with chemical pumps for liquids or booster pump for ejector water). | $Cl_2 + H_2O \rightarrow$ [HOCl / HCl]<br>[HOCl/HCl] + $NaClO_2 \rightarrow$<br>$ClO_{2(g)} + H/OCl^- + NaOH + ClO_3^-$<br>• Low pH<br>• $ClO_3^-$ possible<br>• Relatively slow reaction rates | Excess $Cl_2$ or acid to neutralize NaOH.<br>Production rates limited to ~ 1000 lb/day.<br>High conversion but yield only 80-92%<br>More corrosive effluent due to low pH (~2.8-3.5).<br>Three chemical systems pump HCl, hypochlorite, chlorite, and dilution water to reaction chamber. |
| RECYCLED AQUEOUS CHLORINE OR "FRENCH LOOP"™<br>(Saturated $Cl_2$ solution via a recycling loop prior to mixing with chlorite solution.) | $2HOCl + 2NaClO_2 \rightarrow 2ClO_2 + Cl_2 + 2NaOH$<br>• Excess $Cl_2$ or HCl needed due to NaOH formed. | Concentration of ~3 g/L required for maximum efficiency.<br>Production rate limited to ~ 1000 lb/day.<br>Yield of 92-98% with ~10% excess $Cl_2$ reported. Highly corrosive to pumps; draw-down calibration needed. Maturation tank required after mixing. |
| GASEOUS CHLORINE-CHLORITE<br>(Gaseous $Cl_2$ and 25% solution of sodium chlorite; pulled by ejector into the reaction column.) | $Cl_{2(g)} + NaClO_{2(aq)} \rightarrow ClO_{2(aq)}$<br>• Neutral pH<br>• Rapid reaction<br>• Potential scaling in reactor under vacuum due to hardness of feedstock. | Production rates 5-120,000 lb/day.<br>Ejector-based, with no pumps. Motive water is dilution water. Near neutral pH effluent. No excess $Cl_2$. Turndown rated at 5-10X with yield of 95-99%. Less than 2% excess $Cl_2$. Highly calibrated flow meters with min. line pressure ~ 40 psig needed. |
| GASEOUS CHLORINE-SOLIDS CHLORITE MATRIX<br>(Humidified $Cl_2$ gas is pulled or pumped through a stable matrix containing solid sodium chlorite.) | $Cl_{2(g)} + NaClO_{2(s)} \rightarrow ClO_{2(g)} + NaCl$<br>• Rapid reaction rate<br>• New technology | $Cl_2$ gas diluted with $N_2$ or filtered air to produce ~8% gaseous $ClO_2$ stream. Infinite turndown is possible with >99% yield. Maximum rate to ~1200 lb/day per column; ganged to >10,000 lb/day. |
| ELECTROCHEMICAL<br>(Continuous generation of $ClO_2$ from 25% chlorite solution recycled through electrolyte cell) | $NaClO_{2(aq)} \rightarrow ClO_{2(aq)} + e^-$<br>• New technology | Counter-current chilled water stream accepts gaseous $ClO_2$ from production cell after it diffuses across the gas permeable membrane. Small one-pass system requires precise flow for power requirements (Coulombs law). |
| ACID/PEROXIDE/CHLORIDE | $2NaClO_3 + H_2O_2 + H_2SO_4 \rightarrow 2ClO_2 + O_2 + NaSO_4 + H_20$ | Uses concentrated $H_2O_2$ and $H_2SO_4$.<br>Downscaled version; Foam binding; Low pH. |

Source: Adapted from Gates, 1998.

### 4.1.1.1 Commercial Generators

The conventional chlorine-chlorite solution method generates chlorine dioxide in a two-step process. First, chlorine gas is reacted with water to form hypochlorous acid and hydrochloric acid. These

acids then react with sodium chlorite to form chlorine dioxide. The ratio of sodium chlorite to hypochlorous acid should be carefully controlled. Insufficient chlorine feed will result in a large amount of unreacted chlorite. Excess chlorine feed may result in the formation of chlorate ion, which is an oxidation product of chlorine dioxide and not currently regulated.

***Acid-Chlorite Solution*** - Chlorine dioxide can be generated in direct-acidification generators by acidification of sodium chlorite solution. Several stoichiometric reactions have been reported for such processes (Gordon et al., 1972). When chlorine dioxide is generated in this way, hydrochloric acid is generally preferred (Reaction [1c]).

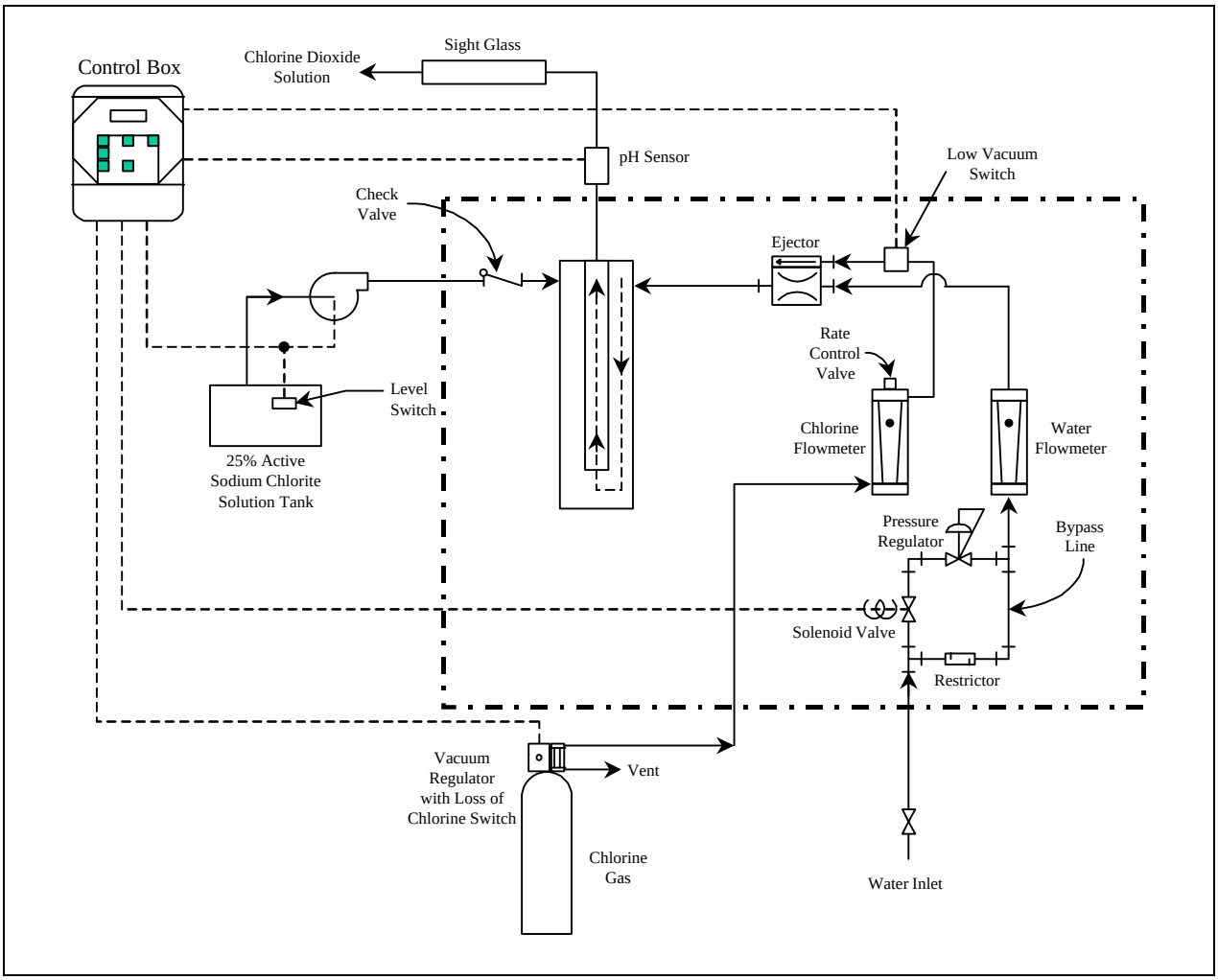
***Aqueous Chlorine-Chlorite Solution*** - Chlorite ion (from dissolved sodium chlorite) will react with hydrochloric acid and hypochlorous acid to form chlorine dioxide in these systems, commonly referred to as conventional systems (Reaction [1b]):

Figure 4-1 shows a typical chlorine dioxide generator using aqueous chlorine-chlorite solution (Demers and Renner, 1992).

If chlorine gas and chlorite ion are allowed to react under ideal conditions (not usually formed in aqueous chlorine type systems), the resulting pH of the effluent may be close to 7. To fully utilize sodium chlorite solution, the more expensive of the two ingredients, excess chlorine is often used. This approach lowers the pH and drives the reaction further toward completion. The reaction is faster than the acid-chlorite solution method, but much slower than the other commercial methods described in the following discussion.

***Recycled Aqueous Chlorine or "French Loop"™*** - In this aqueous chlorine design, shown in Figure 4-2, chlorine gas is injected into a continuously circulating water loop. This eliminates the need for a great excess of $Cl_2$ gas to be fed to the generator since the molecular chlorine will dissolve in the feed water, and thus maintain a low pH level of the feed water. Loop-based generators keep chlorine at or above saturation levels. The low pH condition results in high yields of chlorine dioxide (greater than 95 percent at design production rate) (Thompson, 1989). Chlorine in the generator effluent may react with chlorine dioxide to form chlorate if allowed to stand in batch storage too long. The "French Loop" type of generator is more difficult to operate due to system start-up and control of sodium chlorite feed rate (meter pump), chlorine feed rate (rotameter), and the recirculating loop (pump). Newer designs incorporate a second batching tank for continuous aqueous chlorine storage, thus removing many of these startup or recycling difficulties.

***Gaseous Chlorine-Chlorite Solution*** - Sodium chlorite solution can be "vaporized" and reacted under vacuum with molecular gaseous chlorine. This process uses undiluted reactants and is much more rapid than chlorine solution:chlorite solution methods (Pitochelli, 1995). Production rates are more easily scaled up, and some installed systems have reported producing more than 60,000 pounds per day.




Source: Demers and Renner, 1992.

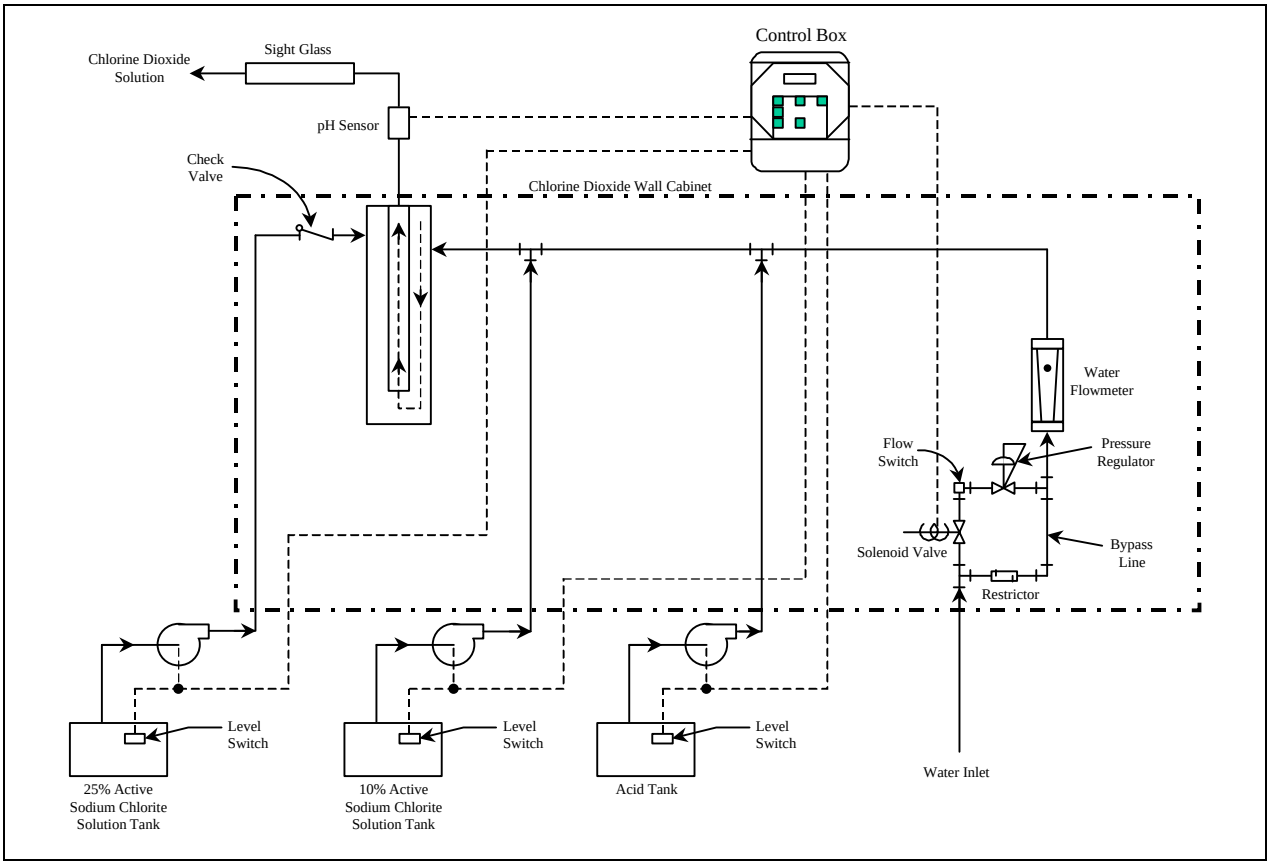
**Figure 4-1. Conventional Chlorine Dioxide Generator When Using Chlorine-Chlorite Method**

The acid-sodium hypochlorite-sodium chlorite method of generating chlorine dioxide is used when chlorine gas is not available. First, sodium hypochlorite is combined with hydrochloric or another acid to form hypochlorous acid. Sodium chlorite is then added to this reaction mixture to produce chlorine dioxide.

### *4.1.1.2 pH Effects on Chlorine Dioxide Generation*

If hypochlorous acid is formed, one of the byproducts of its reaction with sodium chlorite in solution is sodium hydroxide. Since sodium hydroxide is also a common stabilizer of sodium chlorite feedstock, the resulting pH of the mixture can be too high. A high pH slows the formation of chlorine dioxide and impels less efficient chlorate-forming reactions. This is the same process in which chlorite and hypochlorite ions react in drinking water to form chlorate ion. This neutralizing effect of caustic may be influenced by different stabilities used in each of the types and sources of

sodium chlorite which are approved for use in drinking water under AWWA Standard B303-95 (AWWA, 1995).




Source: Demers and Renner, 1992.

**Figure 4-2. Chlorine Dioxide Generation Using Recycled Aqueous Chlorine Method**

In very low pH aqueous chlorine solutions, chlorous acid (and not the chlorite ion) may be directly oxidized to chlorine dioxide as shown in reaction [1d]. At this low pH, gaseous chlorine remains "dissolved" in the water at concentrations higher than the normal occurrence, and allows reaction [1a] to proceed.

$$2HClO_2 + HOCl = HCl + H_2O + 2ClO_2 \qquad [1d]$$

### 4.1.1.3 Chlorate Byproduct Formation

One of the most undesirable byproducts in generators is the chlorate ion ($ClO_3^-$). Chlorate production is possible through reactions with the intermediate dimer, $\{Cl_2O_2\}$. Rather than the chlorite ion being simply "converted" to chlorine dioxide, reactions [1a] through [1d] can result in the supposed formation of the unstable, unsymmetrical intermediate dimer, $\{Cl_2O_2\}$ or $\{Cl^-\text{-}ClO_2\}$ as shown in reaction [2] (Emmenegger and Gordon, 1967).

$$Cl_2 + ClO_2^- = \{Cl\text{-}ClO_2\} + Cl^- \qquad [2]$$

In some generators that operate with relatively low initial reactant concentrations, a significant amount of chlorate is formed by reactions with $\{Cl_2O_2\}$, as shown in reactions [3a], [3b], and [3c].

$$\{Cl_2O_2\} + H_2O = ClO_3^- + Cl^- + 2H^+ \quad [3a]$$

$$\{Cl_2O_2\} + HOCl = ClO_3^- + Cl^- + H^+ \quad [3b]$$

$$\{Cl_2O_2\} + 3HOCl + H_2O = 2ClO_3^- + 5H^+ + 3Cl^- \quad [3c]$$

Highly acidic (pH <3) reaction mixtures force the degradation of $\{Cl_2O_2\}$ to chlorate rather than chlorine dioxide, as well as the direct oxidation of chlorite to chlorate.

The overall reactions that describe chlorate ion formation are:

$$ClO_2^- + HOCl = ClO_3^- + Cl^- + H^+ \quad [4a]$$

and

$$ClO_2^- + Cl_2 + H_2O = ClO_3^- + 2Cl^- + H^+ \quad [4b]$$

The following conditions may also produce the chlorate ion:

- Excessively high ratios of $Cl_2$ gas:$ClO_2^-$.
- Presence of high concentrations of free chlorine at low pH in aqueous solutions.
- Dilute chlorite solutions held at low pH.
- Base-catalyzed disproportionation of chlorine dioxide at high pH values (pH >11).
- Reaction mixtures that are highly acidic (pH <3).
- An excess of hypochlorous acid will directly oxidize chlorite ions to chlorate ions rather than to chlorine dioxide (independent of the rapid formation of the $\{Cl_2O_2\}$ intermediate).

## 4.1.4 Generator Design

As hypochlorous acid is formed under acidic conditions, the lowering of optimal concentrations of precursor reactants will also increase chlorate levels in the generator by promoting reaction [3b]. Therefore, if weak precursor feed stocks or high amounts of dilution water are added to the generator, chlorate will be more prevalent (according to reaction [3a]). These limitations explain why generators most often use ~25 percent chlorite solutions and gaseous (or near-saturated aqueous) chlorine. Higher strength solutions of sodium chlorite (e.g., 37 percent) also are more susceptible to crystallization or stratification at ambient temperatures as high as 25°C(78°F).

Due to these dilution effects, some systems function best as "intermittent batch" generators, (that produce high concentrations of chlorine dioxide) rather than as "continuous" generators (that produce lower concentrations (< 1g/L) of chlorine dioxide). The stored solutions are pumped or injected from

the storage tank. Cycling frequently avoids long-term (over 24 hour periods) storage of the generated solution.

Chlorine loop-type systems can obtain high conversion rates if excess chlorine is always present. Excess chlorine permits the molecular chlorine reaction mechanism (described above) to proceed. The low pH of the mixture also minimizes the contribution of $OH^-$ formed via equation [1b] by neutralizing it. These solutions may still be contaminated with excess chlorine needed to drive the conversion of chlorite ion, but not to the same degree as found in simple aqueous chlorine systems when operated under dilute conditions. Chlorine-loop generators run best at high capacity since the chlorite ion is most available in this production mode.

Conventional or acid-enhanced generators produce chlorine dioxide through the intermediate $\{Cl_2O_2\}$ as long as relatively high concentrations of reactants (~above 20–30 g/L) are maintained in the reaction chamber prior to dilution. Vapor-phase, recycled loop, and solid chlorite-type generators that minimize dilute aqueous reaction conditions can obtain high efficiencies by preventing any chlorite ion from reacting in the "slower" steps described above. This is accomplished by establishing conditions that force the immediate reaction between chlorite ions and gas-phase or molecular chlorine at a rate hundreds of times faster than the $Cl_2$ hydrolysis in water. This essentially minimizes the impact of competitive chlorine hydrolysis or acidification on the dominant [$ClO_2^-$:$Cl_2$ gas] mechanism, and prevents the chlorite ion reacting with hypochlorous acid directly.

In all generators, large excess amounts of $Cl_2$ may result in the over-oxidization of chlorite and directly form chlorate in aqueous solution (reaction [4b]). Precursor chemical feed rates for the generators should always be adjusted to chart settings supplied with generators, notably with the continuous flow, direct gas injection systems. Re-calibration of these systems is sometimes needed on-site if feed stock sodium chlorite is not of the correct strength, or if pre-calibrated flow devices have been replaced.

If aqueous chlorine solutions are mixed with sodium chlorite feed stock solutions, the following mechanisms are dominant, which may affect the formation rates of chlorine dioxide:

- Chlorine gas reacts with water to form hypochlorous and hydrochloric acids, rather than directly with chlorite to form chlorine dioxide. (Water and chlorite both compete for the $Cl_2$ molecule simultaneously) (see equations [4.1a-c] and Section 6.1.1).
- Chlorate ion is formed (reactions [3a], [3b], and [3c]).
- Only 4 moles of chlorine dioxide are obtained from 5 moles of sodium chlorite via direct acidification (reaction [1c]). This may become important at low pH and high chloride ion levels.

The practical side of all of this is that different generators operate under different optimal conditions. For example, reactor columns should not be continuously flooded with excess water in vapor-phase systems. It is the main reason why dry chlorite-based generator reactor columns should not get wet. Over-dilution of the precursor reactants themselves will lower conversion efficiencies due to the

favored formation of chlorate over chlorine dioxide. Batch-type generation should always be carried out at maximal $ClO_2$ concentration with appropriate adjustments at the pump (located downstream of the reactor at the batch tank) for dosage or flow. Changes in chlorine dioxide concentrations in the batch tanks would then be minimized, and pump calibration does not need to include a broad range of chlorine dioxide levels. For the newer gas chlorine generators using dry sodium chlorite in an inert matrix, small amounts of humidifying water in the mixture do not interfere significantly with the simple gaseous $Cl_2$:$ClO_2^-$ reaction. These small traces of water allow for continuous exposure of $ClO_2^-$ on the inert surfaces within the reactor column.

Chlorine dioxide generators are relatively simple mixing chambers. The reactors are frequently filled with media (Teflon™ chips, ceramic or raschig rings) to generate hydraulic turbulence for mixing. A sample petcock valve on the discharge side of the generator is desirable to allow for monitoring of the generation process.

The *Recommended Standards for Water Works* (GLUMRB, 1992) and drinking water design textbooks such as *Unit Processes in Drinking Water Treatment* by Masschelein (1992) are excellent sources for chlorine dioxide generation design criteria and application.

## 4.1.5 Chemical Feed Systems

Fiberglass Reinforced vinyl ester Plastic (FRP) or High Density Linear Polyethylene (HDLPE) tanks with no internal insulation or heat probes are recommended for bulk storage of 25 to 38 percent solution sodium chlorite. Nozzles should include truck unloading vents and local level and temperature indication. Transfer pumps should be centrifugal with 316 stainless steel, fiberglass, Hypalon™, wetted Teflon™ parts, or epoxy resins. The pump should be sealless or equipped with double mechanical seals. The recommended piping material is CPVC, although vinyl ester or Teflon ™ piping systems are acceptable. Carbon steel and stainless steel piping systems are not recommended.

Depending upon system size, sodium chlorite can be purchased in 55-gallon drums, 275-gallon non-returnable totes, or in bulk quantities. A 30-day storage supply of sodium chlorite can easily be met for most small systems by using 55-gallon drums. A 55-gallon drum weighs approximately 600 lbs. Equipment should be provided such that one person can easily handle a drum. All gaseous chlorine or hypochlorite solution-related plumbing should follow Chlorine Institute directives.

Storage and chlorine dioxide systems typically include the following:

- Storage and feeding in a designated space.
- Use of non-combustible materials such as concrete for construction.
- Storage in clean, closed, non-translucent containers. Exposure to sunlight, UV light, or excessive heat will reduce product strength.

- Avoid storage and handling of combustible or reactive materials, such as acids or organic materials, in the sodium chlorite area.
- Secondary containment for storage and handling areas to handle the worse case spill with sumps provided to facilitate recovery.
- A water supply near storage and handling areas for cleanup.
- Inert material should be used in contact with the strong oxidizing and/or acid solutions involved in chlorine dioxide systems.
- Storage tanks with vents to outside.
- Adequate ventilation and air monitoring.
- Gas masks and first aid kits outside of the chemical areas.
- Reactor with glass view ports if it is not made of transparent material.
- Flow monitoring on all chemical feed lines, dilution water lines, and chlorine dioxide solution lines.
- Dilution water should not be excessively hard in order to avoid calcium deposits and should be near neutral pH.
- On-site and frequent testing of chemical solution strengths should be practiced to achieve efficient process control.
- Air contact with chlorine dioxide solutions should be controlled to limit the potential for explosive concentrations possibly building up within the generator. Chlorine dioxide concentrations in air higher than 8 to 10 percent volume should be avoided. Two methods can be applied: operation under vacuum or storage under higher positive pressure (45 to 75 psig) to prevent buildup of gas-phase $ClO_2$ in the head space. Bulk storage (batch) tanks containing $ClO_2$ should be suitably vented to atmosphere.

Sodium chlorite solution feed pumps are commonly diaphragm-metering pumps for liquid feed rate control. If centrifugal pumps are used, the only acceptable packing material is Teflon. If lubrication is needed, minimum quantities of fire-resistant lubricants should be used. Pump motors should be totally enclosed, fan-cooled (TEFC) with sealed-for-life bearings. Couplings should be of the greaseless type. Water lines for mechanical seals should have a pressure gauge and throttling valve on the outlet side. Visual means should be provided to verify adequate water flow. Each pump should include a calibration chamber.

Pipes carrying sodium chlorite should be provided sufficient support to minimize risk of overstressing joints. Flexible connections to pumps should also be provided to minimize risk of vibration damage. Pipe should be sloped to drainage points and valved hose connections provided at strategic points for efficient flushing and draining. Service water for flushing feed lines should be introduced only through temporarily connected hoses protected by a backflow preventer. Service water lines should include check valves. Hose connections from service water lines should have a vent valve to release pressure before the hose is disconnected after use.

Flows are frequently measured with magnetic flow meters, mass flow meters, or rotameters for precise control. Provisions should always be made for back-flow prevention. Sodium chlorite is extremely reactive, especially in the dry form, and care should be taken to protect against potentially explosive conditions.

Chlorine dioxide solution concentrations below about 10 g/L will not produce sufficiently high vapor pressures to present an explosion hazard under most ambient conditions of temperature and pressure. In water treatment, chlorine dioxide solution concentrations rarely exceed 4 g/L for temperatures less than 40°C, and treatment levels generally range from 0.1 to 5.0 mg/L. If temperatures exceed 50°C, storage tanks should be suitably vented due to the higher levels of $ClO_2$ possible. As cold service/potable water is typically used as generator dilution water, these conditions are rarely encountered.

### 4.1.6 Generator Power Requirements

Generator power requirements are similar to those for chlorination systems. For all generators (20 to 12,000 lb/day) power can be supplied from 120 VAC single phase, to 480 VAC three phase. Power demand will vary based upon make-up water pressure available to operate the venturi. Fractional horsepower metering pumps are required, based upon system configuration.

## 4.3 Primary Uses and Points of Application for Chlorine Dioxide

The calculation of CT for chlorine dioxide is similar to other disinfectants, with accurate determinations of residual concentrations being a prerequisite for effective disinfection. Primary disinfectant credit is achieved by the residual concentration and the effective contact time. It has been found in practice that because of the volatile nature of the gas, chlorine dioxide works extremely well in plug flow reactors such as pipe lines. It can be easily removed from dilute aqueous solution by turbulent aeration in rapid mix tanks or purging in recarbonation basins. CT credits should not be expected through a filter because the likelihood that no residual remains in the filtered water (DeMers and Renner, 1992). For post CT disinfection credit, chlorine dioxide can be added before clearwells or transfer pipelines. Ample sampling points should be included to allow close monitoring of residual concentrations. It is well known that chlorine dioxide is commonly destroyed by UV in basins exposed to sunlight or bright fluorescent lights. Therefore, protective design elements should be incorporated if such exposure is anticipated.

### 4.3.1 Disinfection

Before chlorine dioxide is selected for use as a primary disinfectant an oxidant demand study should be completed. Ideally, this study should consider the seasonal variations in water quality, temperature, and application points. Table 4-2 shows typical results for a single sample of a demand study completed on a surface water source.

The MRDL for chlorine dioxide is 0.8 mg/L and the MCL for chlorite is 1.0 mg/L per the D/DBP rule. This means that if the oxidant demand is greater than about 1.4 mg/L, chlorine dioxide may not be used as a disinfectant because the chlorite/chlorate ions byproduct, might exceed the maximum level allowed, unless inorganic byproducts (e.g., chlorite) are subsequently removed. There are numerous means to reduce excessive chlorite levels prior to chlorination in conventional water plants.

**Table 4-2. Surface Water Chlorine Dioxide Demand Study Results**

| Dose (mg/L) | Time (min) | $ClO_2$ (mg/L) | $ClO_2^-$ (mg/L) | $ClO_3^-$ (mg/L) |
|---|---|---|---|---|
| 1.4 | 3 | 0.47 | 0.76 | 0.05 |
| | 10 | 0.30 | 0.98 | 0.06 |
| | 20 | 0.23 | 1.08 | 0.07 |
| | 40 | 0.16 | 1.11 | 0.07 |
| | 60 | 0.11 | 1.11 | 0.07 |

Source: DeMers and Renner, 1992.
Note: *Raw water sample, 23°C, 8.5 pH.

Typical dosages of chlorine dioxide used as a disinfectant in drinking water treatment range from 0.07 to 2.0 mg/L. For plants using chlorine dioxide, median concentrations of chlorite and chlorate were found to be 0.24 and 0.20 mg/L, respectively in an EPA survey (USEPA, 1998), the standard is 1.0 mg/L.

## 4.3.2 Taste and Odor Control

A common application of chlorine dioxide in drinking water in the United States has been for control of tastes and odors associated with algae and decaying vegetation. Chlorine dioxide is also effective in destroying taste and odor producing phenolic compounds. The recommended location for application of chlorine dioxide for this purpose will depend on raw water quality, the type of treatment plant and any other purposes for chlorine dioxide addition. In conventional treatment plants, it is recommended that chlorine dioxide be added near the end of or following, the sedimentation basin. If the raw water turbidity is low (for example, less than 10 NTU), chlorine dioxide can be added at the beginning of the plant. Some utilities follow this practice because chlorine dioxide is effective in controlling algae growth in flocculation and sedimentation basins that are exposed to sunlight (DeMers and Renner, 1992). Such application during periods of darkness may be more successful for nuisance algae control.

## 4.3.3 Oxidation of Iron and Manganese

Chlorine dioxide can be used to oxidize both iron and manganese. Chlorine dioxide reacts with the soluble forms of iron and manganese to form precipitates that can be removed through sedimentation and filtration. Chlorine dioxide reduces to chlorite ion in this reaction (Knocke et al., 1993). About 1.2 mg/L of chlorine dioxide is required to remove 1.0 mg/L of iron, and 2.5 mg/L of chlorine dioxide are required to remove 1.0 mg/L of manganese. For high concentrations of iron and manganese, the use of chlorine dioxide is limited to the 1.0 mg/L chlorite/chlorate ion byproduct, as

described before. Ferrous iron may be added prior to conventional coagulation to chemically reduce chlorite ion (to chloride ion) and improve the overall flocculation process.

# 4.4 Pathogen Inactivation and Disinfection Efficacy

For water treatment, chlorine dioxide has several advantages over chlorine and other disinfectants. In contrast to chlorine, chlorine dioxide remains in its molecular form in the pH range typically found in natural waters (Roberts et al., 1980). Chlorine dioxide is a strong oxidant and disinfectant. Its disinfection mechanisms are not well understood, but appear to vary by type of microorganism

## 4.4.1 Inactivation Mechanisms

Gross physical damage to bacterial cells or viral capsids has not been observed at the low concentrations of chlorine dioxide typically used to disinfect drinking water. Therefore, studies have focused primarily on two more subtle mechanisms that lead to the inactivation of microorganisms: determining specific chemical reactions between chlorine dioxide and biomolecules; and observing the effect chlorine dioxide has on physiological functions.

In the first disinfection mechanism, chlorine dioxide reacts readily with amino acids cysteine, tryptophan, and tyrosine, but not with viral ribonucleic acid (RNA) (Noss et al., 1983; Olivier et al., 1985). From this research, it was concluded that chlorine dioxide inactivated viruses by altering the viral capsid proteins. However, chlorine dioxide reacts with poliovirus RNA and impairs RNA synthesis (Alvarez and O'Brien, 1982). It has also been shown that chlorine dioxide reacts with free fatty acids (Ghandbari et al., 1983). At this time, it is unclear whether the primary mode of inactivation for chlorine dioxide lies in the peripheral structures or nucleic acids. Perhaps reactions in both regions contribute to pathogen inactivation.

The second type of disinfection mechanism focuses on the effect of chlorine dioxide on physiological functions. It has been suggested that the primary mechanism for inactivation was the disruption of protein synthesis (Bernarde et al., 1967a). However, later studies reported the inhibition of protein synthesis may not be the primary inactivation mechanism (Roller et al., 1980). A more recent study reported that chlorine dioxide disrupted the permeability of the outer membrane (Aieta and Berg, 1986). The results of this study were supported by the findings of Olivieri et al. (1985) and Ghandbari et al. (1983), which found that the outer membrane proteins and lipids were sufficiently altered by chlorine dioxide to increase permeability.

## 4.4.2 Environmental Effects

Studies have been performed to determine the effect of pH, temperature, and suspended matter on the disinfection efficiency of chlorine dioxide. Following is a summary of the effects these parameters have on pathogen inactivation.

#### 4.4.2.1 pH

In comparison to chlorine, studies have shown that pH has much less effect on pathogen inactivation for viruses and cysts with chlorine dioxide than with chlorine in the pH range of 6 to 8.5. Unlike chlorine, studies on chlorine dioxide have shown the degree of inactivation of poliovirus 1 (Scarpino et al., 1979) and *Naegleria gruberi* cysts (Chen et al., 1984) increase as the pH increases.

The results of studies on *E. coli* inactivation are inconclusive. It has been found that the degree of inactivation by chlorine dioxide increases as pH increases (Bernarde et al., 1967a). However, an earlier study found that the bactericidal activity of chlorine dioxide was not affected by pH values in the range of 6.0 to 10.0 (Ridenour and Ingols, 1947). A recent study on *Cryptosporidium* found that inactivation of oocysts using chlorine dioxide occurred more rapidly at a pH of 8.0 than 6.0. At a similar CT value, the level of inactivation at pH of 8.0 was approximately twice that at a pH of 6.0 (Le Chevallier et al., 1997). Another study found that chlorine dioxide efficacy increases for *Giardia* inactivation at higher pH levels and that this may be the result of chemical or physical changes in *Giardia* cyst structure rather than pH effects on chlorine dioxide disproportionation (Liyanage et al., 1997). More research is needed to further clarify how pH impacts the effectiveness of chlorine dioxide.

#### 4.4.2.2 Temperature

Similar to chlorine, the disinfection efficiency of chlorine dioxide decreases as temperature decreases (Ridenour and Ingols, 1947). This finding is supported by the data from Chen et al. (1984) shown in Figure 4-3 for the inactivation of *Naegleria gruberi* cysts. The curve shows the CT required to achieve 99 percent inactivation for temperatures between 5 and 30°C.

In a more recent study, LeChevallier et al. (1997) found that reducing the temperature from 20°C to 10°C reduced the disinfection effectiveness of chlorine dioxide on *Cryptosporidium* by 40 percent, which is similar to previous results for *Giardia* and viruses. Gregory et al. (1998) found that even under the most favorable conditions (i.e., at a pH of 8.5), required doses to achieve 2-log *Cryptosporidium* inactivation do not appear to be a feasible alternative, requiring doses of more than 3.0 mg/L with a 60 minute detention time. At neutral pH levels, the required doses may be more than 20 mg/L.

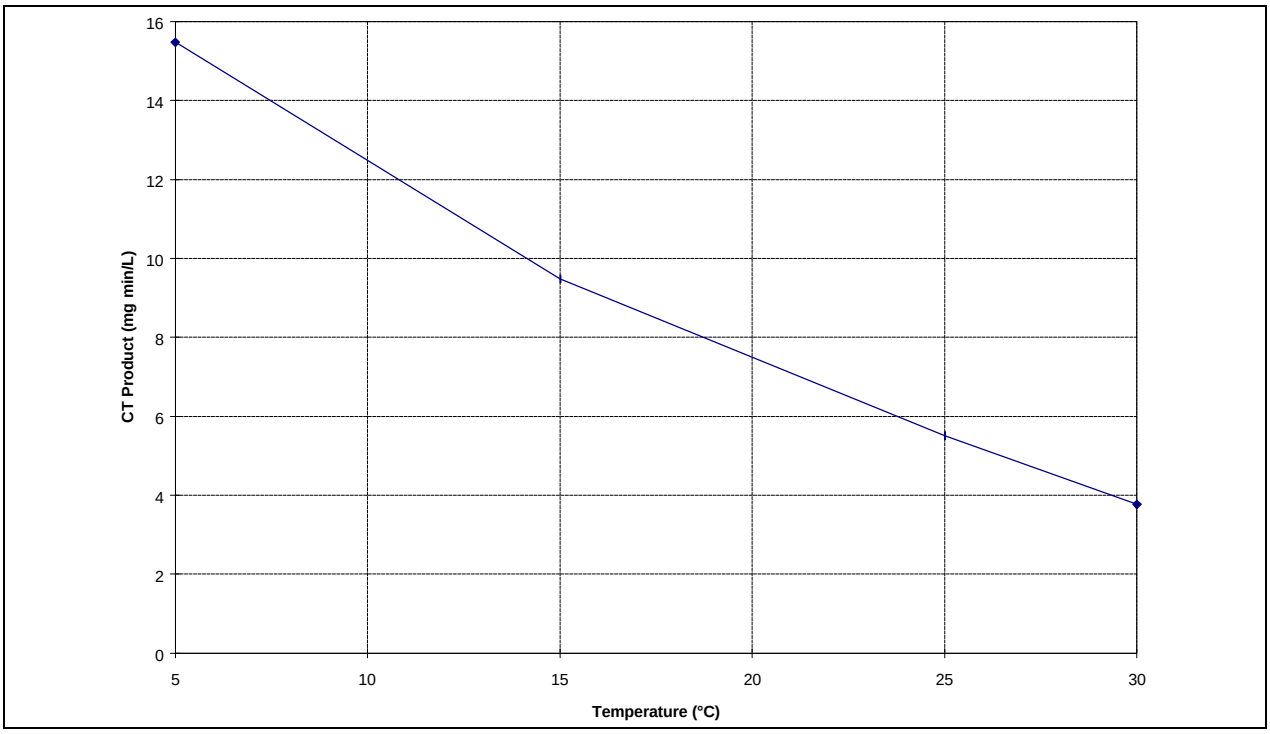



**Figure 4-3. Effect of Temperature on *N. Gruberi* Cyst Inactivation at pH 7**

#### *4.4.2.3 Suspended Matter*

Suspended matter and pathogen aggregation affect the disinfection efficiency of chlorine dioxide. Protection from chlorine dioxide inactivation due to bentonite was determined to be approximately 11 percent for turbidities equal to or less than 5 NTUs and 25 percent for turbidities between 5 and 17 NTUs (Chen et al., 1984).

Laboratory studies of poliovirus 1 preparations containing mostly viral aggregates took 2.7 times longer to inactivate with chlorine dioxide than single state viruses (Brigano et al., 1978). Chen et al. (1984) also found that clumps of *Naegleria gruberi* cysts were more resistant to chlorine dioxide than unclumped cysts or clumps of smaller size.

### 4.4.3 Disinfection Efficacy

Several investigations have been made to determine the germicidal efficiency of chlorine dioxide since its introduction in 1944, as a drinking water disinfectant. Most of the investigations were carried out as a comparison to chlorine; some studies have compared chlorine dioxide and ozone. Chloride dioxide is a more effective disinfectant than chlorine but is less effective than ozone.

#### *4.4.3.1 Bacteria Inactivation*

Quantitative data were published as early as the 1940s demonstrating the efficacy of chlorine dioxide as a bactericide. In general, chlorine dioxide has been determined to be equal to or superior to chlorine on a mass-dose basis. It was demonstrated that even in the presence of suspended matter,

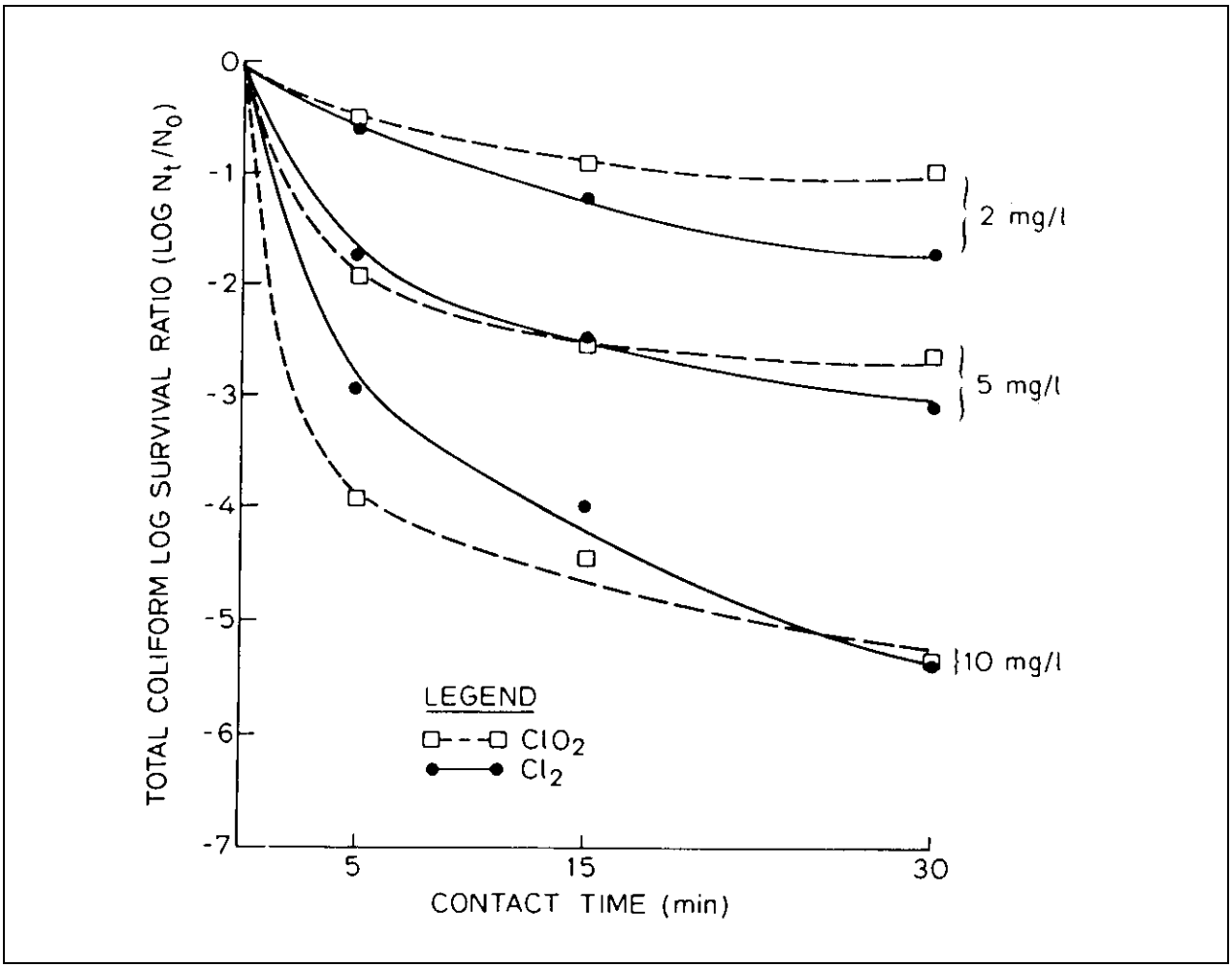
chlorine dioxide was effective against *E. coli* and *Bacillus anthracoides* at dosages in the range of 1 to 5 mg/L (Trakhtman, 1949). Ridenour and Armbruster (1949) reported that an orthotolidine arsenite (OTA) chlorine dioxide residual of less than 1 mg/L was effective against *Eberthella typhosa, Shigella dysenteriae,* and *Salmonella paratyphi* B. Under similar pH and temperature slightly greater OTA residuals were required for the inactivation of *Pseudomonas aeruginosa* and *Staphylococcus aureus.*

Chlorine dioxide was shown to be more effective than chlorine at inactivating *B. subtilis, B. mesentericus,* and *B. megatherium* spores (Ridenour et al., 1949). Moreover, chlorine dioxide was shown to be just as effective or more effective than chlorine at inactivating *Salmonella typhosa* and *S. paratyphi* (Bedulivich et al., 1954).

In the early 1960s several important contributions were made by Bernarde et al. (1967a and 1967b). Chlorine dioxide was found to be more effective than chlorine at disinfecting sewage effluent and the rate of inactivation was found to be rapid.

A comprehensive investigation of chlorine dioxide as disinfectant was performed by Roberts et al. (1980). The investigation was performed using secondary effluents from three different wastewater treatment plants. One of the objectives was to determine the relationships between dosages and contact times and bactericidal efficiency. Dosages were compared for 2, 5, and 10 mg/L of chlorine dioxide and chlorine. The contact times selected were 5, 15 and 30 minutes. Results of the investigation are shown in Figure 4-4. As shown, chlorine dioxide demonstrated a more rapid coliform inactivation than chlorine at the shortest contact time of 5 minutes and higher concentrations. However, after 30 minutes of contact time, chlorine dioxide was equal or slightly less efficient than chlorine as a bactericide.

Oliveri et al. (1984) studied the effectiveness of chlorine dioxide (and chlorine) residuals in inactivating total coliform and f2 coliphage virus in sewage introduced to a water distribution system. Initial chlorine dioxide residuals between 0.85 and 0.95 mg/L resulted in an average 2.8–log inactivation of the total coliform and an average 4.4-log inactivation of the f2 coliphage virus, over a contact time of 240 minutes.




Source: Roberts et al., 1980.

**Figure 4-4. Comparison of Germicidal Efficiency of Chlorine Dioxide and Chlorine**

### *4.4.3.2 Protozoa Inactivation*

The disinfection efficiency of chlorine dioxide has been shown to be equal to or greater than chlorine for *Giardia* inactivation. Based on a 60 minute contact time, chlorine dioxide doses in the range of 1.5 to 2 mg/L are capable of providing a 3-log *Giardia* inactivation at 1°C to 25°C and pHs of 6 and 9 (Hofmann et al., 1997). Depending on the temperature and pH, *Cryptosporidium* has been found to be 8 to 16 times more resistant to chlorine dioxide than *Giardia* (Hofmann et al., 1997). Although some *Cryptosporidium* oocysts remained viable, one group of researchers found that a 30-minute contact time with 0.22 mg/L chlorine dioxide could significantly reduce oocyst infectivity (Peeters et al., 1989). In contrast, other researchers have found that CT values in the range of 60 to 80 mg·min/L were necessary to provide 1- to 1.5-log inactivation (Korich et al., 1990; Ransome et al., 1993). Finch et al. (1995) reported that the CT values for 1-log inactivation was in the range of 27 to 30 mg·min/L. For 2-log inactivation, the CT value was approximately 40 mg·min/L, and 70 mg·min/L for 3-log inactivation. Finch et al. (1997) found 3-log inactivation of *Cryptosporidium* oocysts with initial chlorine dioxide residual concentrations of 2.7 and 3.3 mg/L for contact times of 120 minutes, at pH of 8.0 and a temperature of 22ºC.

Both Chen et al. (1985) and Sproul et al. (1983) have investigated the inactivation of *Naegleria gruberi* cysts by chlorine dioxide. Both studies concluded that chlorine dioxide is an excellent disinfectant against cysts and that chlorine dioxide is better than or equal to chlorine in terms of inactivation. Chlorine dioxide was found to be superior to chlorine at higher pHs. However, the authors cautioned that the CT required for 2-log inactivation was much higher than normally employed for water treatment at that time.

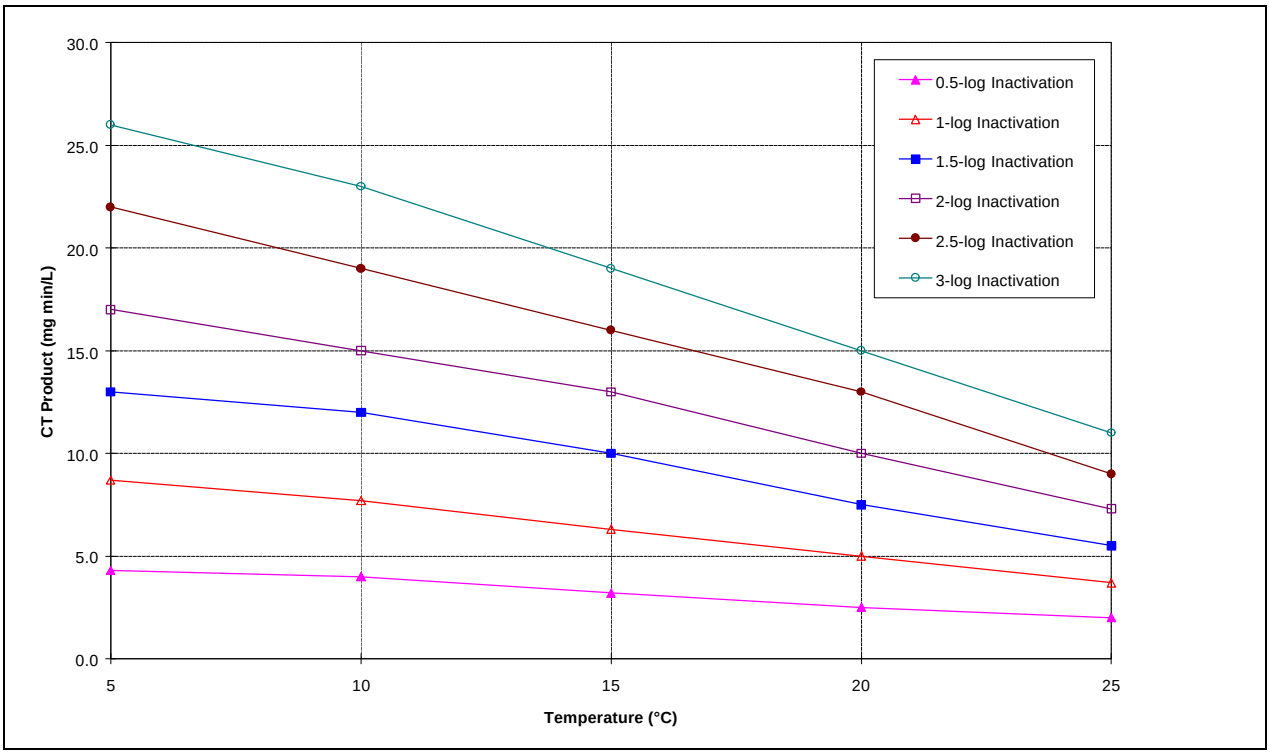
#### 4.4.3.3 Virus Inactivation

Chlorine dioxide has been shown to be an effective viricide. Laboratory studies have shown that inactivation efficiency improves when viruses are in a single state rather than clumped. It was reported in 1946 that chlorine dioxide inactivated *Poliomyelitis* (Ridenour and Ingols, 1946). This investigation also showed that chlorine dioxide and free chlorine yielded similar results. Other studies have verified these findings for poliovirus 1 (Cronier et al., 1978) and *Coxsackie* virus A9 (Scarpino, 1979). At greater than neutral pHs (where hypochlorite ion is the predominant species) chlorine dioxide has been found to be superior to chlorine in the inactivation of numerous viruses such as echovirus 7, coxsackie virus B3, and sendaivirus (Smith and McVey, 1973). Sobsey (1998) determined CT values based on a study of Hepatitis A virus, strain HM 175. The study found 4-log inactivation levels are obtainable at CT values of less than 35 at 5°C and less than 10 at a temperature of 25°C.

#### 4.4.3.4 CT Values

Chlorine dioxide is regarded as a strong disinfectant that is effective at inactivating bacterial, viral, and protozoan pathogens. CT values for *Giardia* and virus inactivation are shown in Figure 4-5 and Figure 4-6, respectively (AWWA, 1991).

CT values shown in Figure 4-5 are based on disinfection studies using in vitro excystation of *Giardia muris*. Average CT values for 2 log removal were extrapolated using first order kinetics and multiplied by a safety factor of 1.5 to obtain the CT values for other log removal CT values. Due to the limited amount of data available at pH values other than 7, the same CT values are used for all pHs. Because chlorine dioxide is more effective at a pH 9 than at a pH of 7, the CT values shown in Figure 4-5 are more conservative for higher pHs than for lower pHs. A lower safety factor was used to derive the CT values for chlorine dioxide than for ozone due to the fact that the chlorine dioxide values were derived from *Giardia muris* studies, which are more resistant than *Giardia lamblia*.

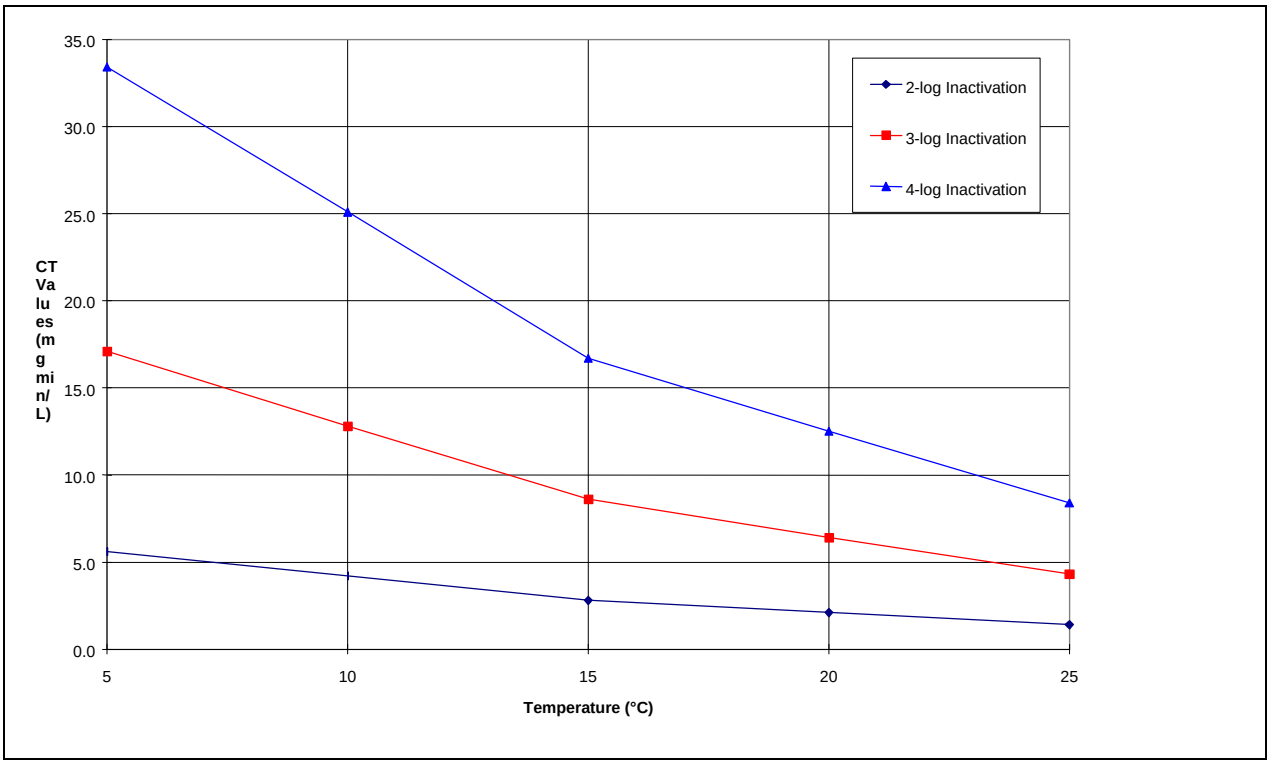



Source: AWWA, 1991.

**Figure 4-5. CT Values for Inactivation of *Giardia* Cysts by Chlorine Dioxide**

CT values shown in Figure 4-6 were obtained by applying a safety factor 2 to the average CT values derived from the studies on hepatitis A virus, strain HM 175 (Sobsey, 1988). CT values at temperatures other than 5°C were derived by applying a twofold decrease for every 10°C increase in temperature.

Figure 4-7 and Figure 4-8 show the relationship between CT products and log inactivation of *Cryptosporidium* at 20 and 10°C, respectively, and pHs of 6 and 8. CT values shown in Figure 4-7 and Figure 4-8 indicate that oocysts were more rapidly inactivated at pH 8 than 6 and that temperature does impact the disinfection efficiency of chlorine dioxide. Reducing the temperature from 20 to 10°C reduced the disinfection effectiveness by 40 percent. Finch (1997) is studying *Cryptosporidium* inactivation under laboratory conditions using a variety of different disinfectants, one of which is chloride dioxide.




Source: AWWA, 1991.

**Figure 4-6. CT Values for Inactivation of Viruses by Chlorine Dioxide**

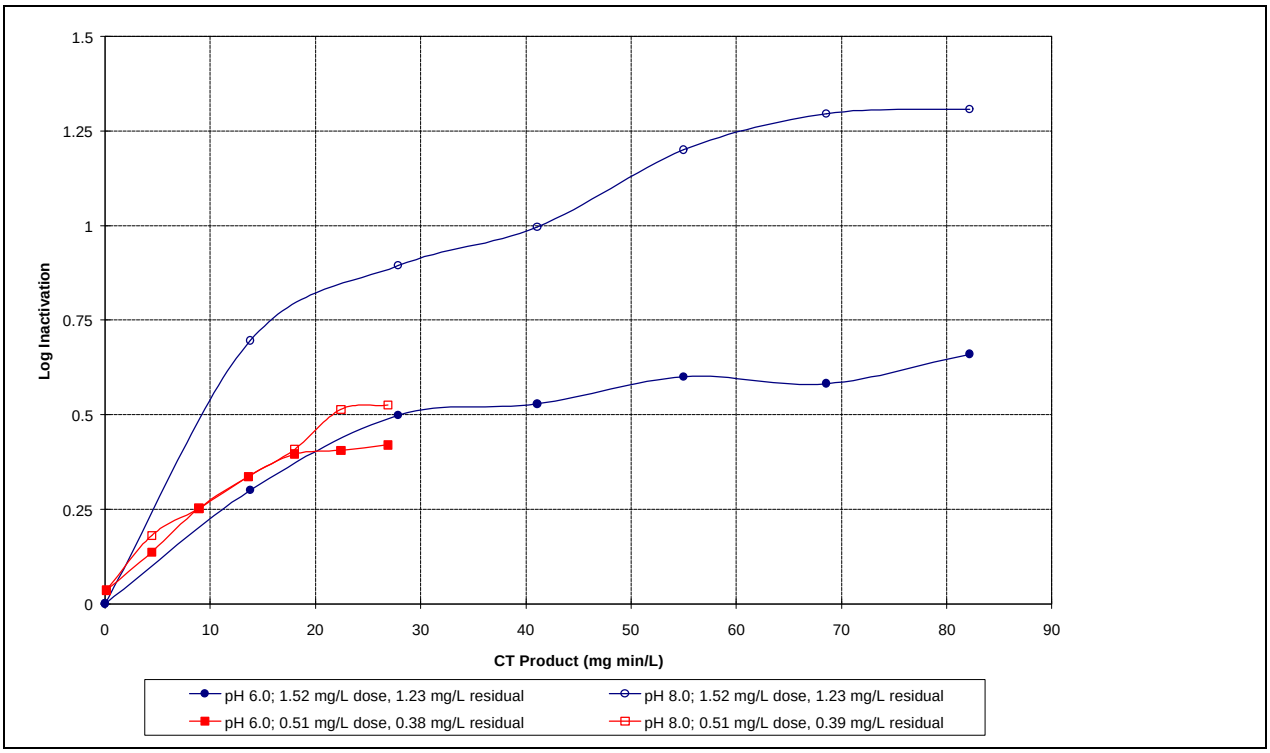
# 4.5 Chlorine Dioxide Disinfection Byproducts

Byproducts from the use of chlorine dioxide include chlorite, chlorate, and organic DBPs. This section discusses the formation of these byproducts and methods to reduce or remove these DBPs. The use of chlorine dioxide aids in reducing the formation of TTHMs and HAAs by oxidizing precursors, and by allowing the point of chlorination to be moved farther downstream in the plant after coagulation, sedimentation, and filtration have reduced the quantity of NOM.

## 4.5.1 Production of Chlorite and Chlorate

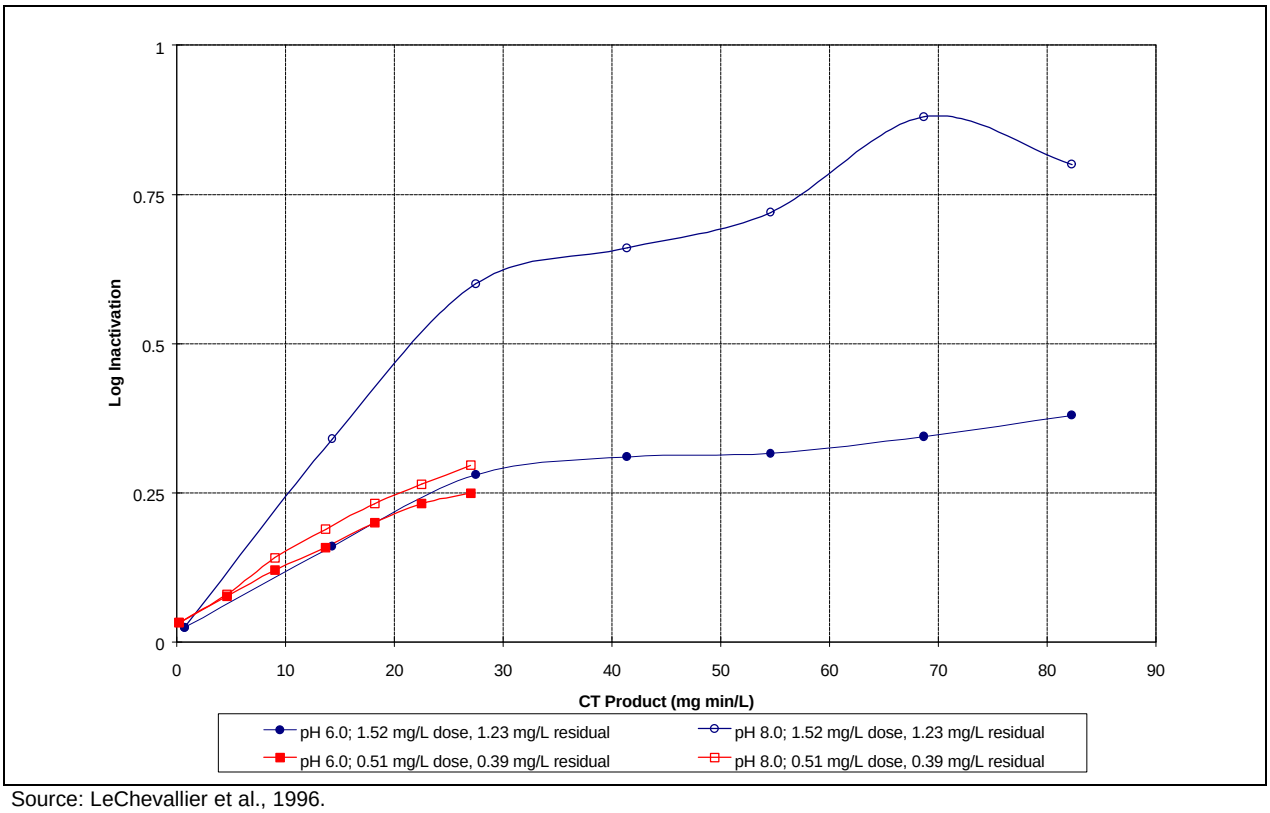
Chlorite and chlorate are produced in varying ratios as endproducts during chlorine dioxide treatment and subsequent degradation. The primary factors affecting the concentrations of chlorine dioxide, chlorite, and chlorate in finished drinking water involve:

- Dosage applied/oxidant demand ratio.
- Blending ratios of sodium chlorite and chlorine during the chlorine dioxide generation process.
- Exposure of water containing chlorine dioxide to sunlight.




Source: LeChevallier et al., 1996.

**Figure 4-7. *C. parvum* Inactivation by Chlorine Dioxide at 20°C**




Source: LeChevallier et al., 1996.

**Figure 4-8. *C. parvum* Inactivation by Chlorine Dioxide at 10°C**

- Reactions between chlorine and chlorite if free chlorine is used for distribution system residual maintenance.
- Levels of chlorate in sodium chlorite feedstock.

Incomplete reaction or non-stoichiometric addition of the sodium chlorite and chlorine reactants can result in unreacted chlorite in the chlorine dioxide feed stream. Dilute chlorine dioxide solutions are stable under low or zero oxidant-demand conditions. The quantity of chlorate produced during the chlorine dioxide generation process is greater with excess chlorine addition. Likewise, a low or high pH can increase the quantity of chlorate during the chlorine dioxide generation process. See Section 4.2, "Generation," for a detailed discussion of the chemistry of chlorine dioxide generation.

Numerous inorganic and biological materials found in raw water will react with chlorine dioxide (Noack and Doerr, 1977). Chloride ($Cl^-$) and chlorite ($ClO_2^-$) ions are the dominant degradation species arising from these reactions, although chlorate ($ClO_3^-$) can appear for a variety of reasons when chlorine dioxide is used (Gordon et al., 1990; Werdehoff and Singer, 1987). The immediate redox reactions with natural organic matter play the dominant role in decay of chlorine dioxide into chlorite in drinking water (Werdehoff and Singer, 1987). Chlorite ion is generally the primary product of chlorine dioxide reduction. The distribution of chlorite and chlorate is influenced by pH and sunlight. Approximately 50 to 70 percent of the chlorine dioxide consumed by oxidation reactions is converted to chlorite under conditions typical in water treatment (Rav-Acha et al., 1984; Werdehoff and Singer, 1987). The application of 2 mg/L chlorine dioxide is expected to produce 1 to 1.4 mg/L of chlorite (Singer, 1992).

Chlorite is relatively stable in the presence of organic material but can be oxidized to chlorate by free chlorine if added as a secondary disinfectant (Singer and O'Neil, 1987).

$$ClO_2^- + OCl^- = ClO_3^- + Cl^-$$

Chlorate is therefore produced through the reaction of residual chlorite and free chlorine during secondary disinfection.

In addition, chlorine dioxide also disproportionates under highly alkaline conditions (pH>9) to chlorite and chlorate according to the following reaction:

$$2ClO_2 + 2OH^- = ClO_2^- + ClO_3^- + H_2O$$

In water treatment processes that require high pH, such as softening, chlorine dioxide should be added after the pH has been lowered (Aieta et al., 1984).

The occurrence of photochemical decomposition of chlorine dioxide can affect the ultimate concentrations of chlorine dioxide, chlorite, and chlorate in water treated with chlorine dioxide. Moreover, generally, sunlight may increase chlorate concentrations in uncovered storage basins containing water with chlorine dioxide residuals. Exposure to ultraviolet light will also change the potential reactions between chlorine dioxide and the bromide ion.

## 4.5.2 Organic DBPs Produced by Chlorine Dioxide

Chlorine dioxide generally produces few organic DBPs. However, Singer (1992) noted that the formation of non-halogenated organic byproducts of chlorine dioxide has not been adequately researched, and expected that chlorine dioxide will produce the same types of oxidation byproducts that are produced through ozonation. The application of chlorine dioxide does not produce THMs and produces only a small amount of total organic halide (TOX) (Werdehoff and Singer, 1987).

A study was conducted in 1994 by Richardson et al., to identify semivolatile, organic DBPs produced by chlorine dioxide treatment in drinking water. Samples were taken from a pilot plant in Evansville, Indiana that included the following treatment variations:

- Aqueous chlorine dioxide;
- Aqueous chlorine dioxide, ferrous chloride, ($FeCl_2$), chlorine ($Cl_2$), and dual media filtration (sand and anthracite);
- Gaseous chlorine dioxide; and
- Gaseous chlorine dioxide, ferrous chloride ($FeCl_2$), chlorine ($Cl_2$), and dual media filtration (sand and anthracite).

Using multispectral identification techniques, more than 40 different DBPs (many at sub-nanogram/L [ng/L] levels) were identified including carboxylic acids and maleic anhydrides isolated from XAD™ concentrates, some of which may be regulated in the Stage 2 DBPR. THMs were not found after chlorine dioxide was added to the water; however, THMs did show up during subsequent chlorination.

## 4.5.3 Chlorine Dioxide DBP Control Strategies

EPA recommends that the total concentration of chlorine dioxide, chlorite, and chlorate be less than 1.0 mg/L as $Cl_2$ (USEPA, 1983). In addition, chlorine dioxide concentrations exceeding 0.4 to 0.5 mg/L contribute to taste and odor problems (AWWA, 1990). Due to these issues, the use of chlorine dioxide to provide a disinfectant residual is somewhat limited in moderate to high TOC water. In low oxidant-demand water, however, $ClO_2$ residuals may last several days.

Once formed, chlorate is stable in finished drinking water. No known treatment exists for removing chlorate once it is formed. However, three strategies (Gallagher et al., 1994) that have been proven effective for chlorite removal are:

- Adding reduced-sulfur compounds such as sulfur dioxide and sodium sulfite (not recommended).
- Applying either granular activated carbon (GAC) or powdered activated carbon (PAC).
- Adding reduced iron salts, such as ferrous chloride and ferrous sulfate.

Chlorite removal from drinking water through sulfur dioxide and other sulfur-based reducing agents has been reported effective, but not desirable. A study of chlorite removal by sulfur dioxide indicates that a lower pH level yields higher chlorite removal, and chlorite removal efficiencies increase as the sulfur dioxide dose increases. Unfortunately, this removal process forms significant levels of chlorate when sulfur dioxide and metasulfite are utilized. Therefore, it is concluded that treatment with sulfur dioxide and metasulfite is not desirable for chlorite removal (Dixon and Lee, 1991). In addition, sodium thiosulfate results in effective chlorite reduction, but the degree of removal is highly dependent upon pH and contact time and relatively high dosages are required. Again, this application of sodium thiosulfate is not desirable because the required dosages are too high (Griese et al., 1991).

The addition of ferrous iron in drinking water is effective for chlorite removal, with chloride the expected byproduct. Chlorite reduction occurs quickly in the pH range of 5 to 7, and complete reduction occurs within 3 to 5 seconds. Excess reduced iron remaining in solution reacts with dissolved oxygen at neutral pH, but under acidic conditions (pH < 6.5) the stability of the soluble iron can create aesthetic (staining) problems if excess iron is used. Special consideration should be given to ferrous iron dosage requirements so that the secondary MCL for iron is not exceeded (Knocke and Iatrou, 1993).

Chlorite can be controlled by PAC at relatively high dosages (10 to 20 mg/L) and low pHs (5.5 to 6.5). Unless PAC is used for other purposes, such as odor control, it requires large doses and is not cost effective. PAC brands can differ in their capacity to reduce chlorite.

GAC can remove chlorite but breakthrough may occur relatively early. The removal of chlorite by GAC appears to be a result of adsorption and chemical reduction (Dixon and Lee, 1991). There is an initial high removal efficiency due to chlorite adsorption. As the adsorptive sites are occupied, chemical reduction on the GAC surface becomes the primary removal mechanism. This results in an initial high removal efficiency. Although chlorite levels exiting the GAC filters are low, the chlorate levels are high, most likely a result of reactions in the GAC filters between chlorite and free chlorine. According to studies, the capacity of GAC beds is low, and if free chlorine and chlorite ion are present in the GAC influent, chlorate ion will form. The most effective way to operate GAC for chlorite reduction and avoid chlorate is to minimize production run times and have no chlorine present in the filter.

## 4.6 Status of Analytical Methods

In addition to the monitoring requirements that apply regardless of the disinfectant used, the DBPR requires that water systems that use chlorine dioxide for disinfection or oxidation must also monitor their system for chlorine dioxide and chlorite.

## 4.6.1 Chlorine Dioxide and Chlorite Analytical Methods

For compliance monitoring for chlorine dioxide, systems must use one of the two methods specified in 40 CFR §141.131(c), including (1) DPD, Standard Method 4500-$CLO_2$ D, or (2) Amperometric Method II, Standard Method 4500-$CLO_2$ E. Where approved by the state, systems may also measure residual disinfectant concentrations for chlorine dioxide by using DPD colorimetric test kits.

For compliance monitoring for chlorite, systems must use one of the three methods specified in 40 CFR §141.131(b), including (1) Amperometric Titration, Standard Method 4500-$CLO_2$ E, (2) Ion Chromatography, EPA Method 300.0, or (3) Ion Chromatography, EPA Method 300.1. The regulations specify that Amperometric Titration may be used for routine daily monitoring of chlorite at the entrance to the distribution system, but that Ion Chromatography must be used for routine, monitoring of chlorate and monthly additional monitoring of chlorate in the distribution system.

Details of these analytical procedures can be found in:

- *Standard Methods for the Examination of Water and Wastewater*, 19th Edition, American Public Health Association, 1995.
- *Methods for the Determination of Inorganic Substances in Environmental Samples*. USEPA. 1993. EPA/600/R-93/100.
- *USEPA Method 300.1, Determination of Inorganic Anions in Drinking Water by Ion Chromatography, Revision 1.0.* USEPA. 1997. EPA/600/R-98/118.

Table 4-3 summarizes the analytical methods approved for use for chlorine dioxide and chlorite and provides some background information for each method.

## 4.6.2 Chlorine Dioxide Monitoring for Systems Using Chlorine Dioxide

For chlorine dioxide monitoring, community, non-transient non-community, and transient non-community water systems that use chlorine dioxide for disinfection or oxidation, are required to take daily samples at the entrance to the distribution system. For any daily sample that exceeds the chlorine dioxide MRDL of 0.8 mg/L, the system must take additional samples in the distribution system the following day at the locations specified in the DBPR, in addition to the daily sample required at the entrance to the distribution system.

Additional sampling is to be performed in one of two ways, depending on the disinfectant that is used to maintain a disinfectant residual in the distribution system. If chlorine dioxide or chloramines are used to maintain a disinfectant residual, or if chlorine is used to maintain the residual and there are no disinfection addition points after the entrance to the distribution system (i.e., no booster chlorination), the system must take three samples as close to the first customer as possible, at intervals of at least six hours. If chlorine is used to maintain a disinfectant residual and there are one or more disinfection addition points after the entrance to the distribution system, the system must take one sample at each of the following locations: (1) as close to the first customer as possible, (2) in a location representative of average residence time,

and (3) as close to the end of the distribution system as possible (reflecting maximum residence time in the distribution system). Chlorine dioxide monitoring may not be reduced.

If any daily sample taken at the entrance to the distribution system exceeds the MRDL, and on the following day one (or more) of the three samples taken in the distribution system exceed the MRDL, the system is in violation of the MRDL. The system must take immediate corrective action to lower the level of chlorine dioxide below the MRDL, and must notify the public of the acute violation pursuant to 40 CFR §141.32. The system must also report to the State pursuant to 40 CFR §141.134.

If any two consecutive daily samples taken at the entrance to the distribution system exceed the MRDL, the system is also in violation of the MRDL and must notify the public of the non-acute violation pursuant to 40 CFR §141.32. The system must also report to the State pursuant to 40 CFR §141.134.

### 4.6.3 Chlorite Monitoring for Systems Using Chlorine Dioxide

For chlorite monitoring, community and non-transient non-community water systems that use chlorine dioxide for disinfection or oxidation are required to take daily samples at the entrance to the distribution system. For any daily sample that exceeds the chlorite MCL of 1.0 mg/L, the system must take additional samples in the distribution system the following day at the locations specified in the DBPR. These additional samples are to be collected at: (1) a location as close to the first customer as possible, (2) a location representative of average residence time, and (3) a location as close to the end of the distribution system as possible (reflecting maximum residence time in the distribution system).

In addition, systems using chlorine dioxide must take a three-sample set each month in the distribution system similar to the three locations required if the chlorite MCL is exceeded in the sample collected at the entrance to the distribution system. Specifically, these three-sample sets are to be collected: (1) in a location near the first customer, (2) in a location representative of average residence time, and (3) at a location reflecting maximum residence time in the distribution system. Any additional routine sampling must be conducted in the same three-sample sets at the specified locations. This monthly sampling requirement may be reduced to quarterly after one year of monitoring where: (1) no individual chlorite sample taken in the distribution system has exceeded the MCL and (2) the system has not been required to conduct follow-up monitoring as a result of a daily sample collected at the entrance to the distribution system. These systems can remain on an annual schedule until either the daily sample or any of the three individual quarterly samples exceed the MCL, at which time, the system must revert to monthly monitoring.

If the arithmetic average of any three-sample set exceeds the chlorite MCL of 1.0 mg/L, the system is in violation of the MCL and must notify the public pursuant to 40 CFR §141.32, in addition to reporting to the State pursuant to 40 CFR §141.134.

## 4.7 Operational Considerations

As with all disinfectant selections, the primacy agency should be consulted when selecting disinfectants. Certain states have their own operational, maintenance, and monitoring requirements

for the application of chlorine dioxide. California prohibits the use of chlorine dioxide in ground water systems, according to Merkle et al., 1997. Also, in Texas, the Texas Natural Resources Conservation Commission (TNRCC) requires the public water supply to sign a bilateral agreement which outlines a detailed operator qualifications requirement, testing methods, and procedures, monitoring locations, testing frequency and reporting procedures. The chlorine dioxide concentration leaving the water treatment plant must be less than 0.8 mg/L and the chlorite concentration in the distribution system must be less than 1.0 mg/L.

State requirements must be reviewed to determine the cost-effectiveness of utilizing chlorine dioxide as part of the overall water treatment scheme. Analytical testing and reporting requirements may have significant labor and cost impacts.

**Table 4-3. Analytical Methods for Chlorine Dioxide and Related Compounds**

| Method | Basis | Interferences | Limits |
|---|---|---|---|
| DPD as Test Kits Colorimetric (SM-4500-$ClO_2$.G) | Colored oxidation product. Use of color comparator is not recommended. Use instrument detection. | $Mn_2^+$, other $Cl_2$, related oxidants. | > 0.1 mg/L |
| DPD-glycine Method Colorimetric (SM 4500- $ClO_2$.D) | Colored product, free $Cl_2$ is masked with glycine as chloraminacetic acid. | $ClO_2^-$ slowly; other oxidants. | > 0.1 mg/L |
| DPD-FAS Titrimetric method (SM 4500- $ClO_2$.D) | DPD color titration with standard FAS until red color is discharged. | Iron, other oxidants. | > 0.1 mg/L |
| 5-Step Amperometric Method 4500-$ClO_2$.E | $I^-$ oxidation; pH control and gas purging steps. Skilled analyst needed. | Suitable for $ClO_2$ generated solution. Low levels not okay. | ~ PQL $ClO_2^-$: 0.1-0.0.5 mg/L; $ClO_3^-$ at 0.5 mg/L |
| Ion Chromatography (EPA Method 300.0 or 300.1) Conductivity | Must use AS9 column, ext. standards & suppression. | No other oxidants. Chloramines, $ClO_2$; $OCl^-$ & HOCL undetectable. | ~ 0.05 mg/L |
| Two-step Amperometric Method 4500-$ClO_2$.E | $I^-$ Oxidation; pH control. Amendable to operator-based dosage control. Practical method. | $Cu_2$+, $Mn_2$+, $NO_2^-$ Accounts for free $Cl_2$, $NH_2Cl$, $ClO_2^-$ species. | > 0.1 mg/L, not $ClO_3^-$ |

Source: Gates, 1998.
Notes: SM = Standard Methods

## 4.7.1 Process Considerations

The basic components of chlorine dioxide generation systems include:

- Aqueous hypochlorite solution storage and feed system;
- Sodium chlorite storage and feed system;
- Acid storage and feed system (for Direct-Acidification generators);
- Chlorine storage and feed system;
- Chlorine dioxide generator; and
- Chlorine dioxide feed piping and dispersion equipment.

Sodium chlorite storage and feed systems are basically liquid systems that consist of a storage tank(s) and solution feed pumps. Outside storage of 25 percent solutions (or greater) of sodium chlorite is not recommended in cold climates since stratification may occur below 4°C (40°F). Any ice formation may also damage the storage tanks. In some cases, storage might be separated into bulk tanks and smaller operational or day tanks that are filled periodically. Storage of dark drums for long periods in hot climates should be avoided since sodium chlorite decomposition will occur. In the storage area, light fixtures, switches, wiring, and conduit runs should be located to avoid the risk of sodium chlorite spilling on them.

## 4.7.2 Generator Operation

A manual chlorine dioxide feed system may be used where the chlorine dioxide dose remains fairly constant. The reagent chemicals are manually set for the desired chlorine dioxide capacity at a ratio of chemicals optimized for maximum chlorine dioxide yield. Some generating systems can produce 95 percent pure chlorine dioxide solutions at full design capacity, but purity can vary when the feed rate is changed. Turndown capacity may be limited by precision of the flow metering devices, typically 20 percent of rate capacity. Purity can vary when the feed rate is changed significantly. Feed water alkalinity, operating conditions, and pH also can affect yield. The ratio of reagent chemicals should be routinely adjusted for optimum operation. Chlorine dioxide generators can be provided with automated control to provide modulation of chlorine dioxide feed rates based upon changes in flow (flow paced) and chlorine dioxide demand (residual control). The automatic modulation of the generators to meet a demand setpoint varies with manufacturer. Generally, vacuum and combination systems are limited by the hydraulic requirements of the venturi and the optimum reaction conditions for chlorine dioxide generation. A chemical metering pump or injector system is then used with a batch production system to control the applied dose of chlorine dioxide.

## 4.7.3 Feed Chemicals

Chlorine dioxide is generated when sodium chlorite is either oxidized or acidified, or both, under controlled pH and temperature conditions. Commonly, solutions of 25 percent active sodium chlorite or less are used in chlorine dioxide generators. The major safety concern for solutions of sodium chlorite is the unintentional and uncontrollable release of high levels of chlorine dioxide. Such levels may approach detonation or conflagration concentrations by accidental acidification.

The feedstock acid used by some of the generators is only one source of accidental chemical acidification. Accidental mixing with large amounts of any reducing agent or oxidizable material (such as powdered activated carbon or flammable solvents) also represents a significant hazard. The AWWA Standard B303-95 (a) includes an outline of some of these materials (AWWA, 1995).

Another concern when handling and storing sodium chlorite solutions is crystallization, which occurs as a result of lower temperatures and/or higher concentrations. Crystallization will plug pipelines, valves, and other equipment. Sodium chlorite solution should not be allowed to evaporate to a powder. If dried, this product becomes a fire hazard and can ignite in contact with combustible materials. A sodium chlorite fire may result in a steam explosion if too much water and inappropriate fire-fighting techniques are used to quench such a fire. As the temperature of burning sodium chlorite is around 2200°C, water quickly turns to steam. Because thermal breakdown products of sodium chlorite at high temperatures include molecular oxygen, appropriate techniques are required to correctly extinguish closed containers or large amounts of dry material that has been ignited.

Stratification of sodium chlorite in holding tanks may also occur and would influence the chlorine dioxide yield. If stratification occurs in the bulk tank, sodium chlorite changes from high density to

low density as it is fed. The density will continue to change until the material is re-mixed. In stratified tanks, excess chlorite would be fed to the generator since the bottom of the tank will have denser material, and this material would have more chlorite than required. Similarly, the bulk tank would later discharge too little chlorite.

Although infrequent, such stratification is not readily apparent and may likely remain unnoticed by operations unless the generator performance is evaluated frequently. If stratification or crystallization occurs in bulk delivery trucks, the entire content should be warmed prior to delivery so that the sodium chlorite is re-mixed. Operators should be aware of the possibility of stratification and crystallization during delivery conditions.

Sodium chlorite is commercially available as a 38 percent or 25 percent solution. Chemical and physical properties are given in Table 4-4.

**Table 4-4. Properties of Sodium Chlorite as Commercially Available**

| | 38% Solution* | 25% Solution* |
|---|---|---|
| Sodium Chlorite, (%) $NaClO_2$ | 38 | 25 |
| Sodium Chloride, (%) NaCl | 1.5-7.5 | 1-4.5 |
| Inert Ingredients, mixture of other sodium salts (%) | 3-4 | 3-4 |
| Water (%) | 55-61 | 68-74 |
| Appearance | Slightly cloudy, pale yellow | Clear, pale yellow |
| Density @ 35°C (lb/gal), typical | 11.4 | 10.1 |
| Crystallization Point (°C) | 25 | -7 |

* Source: Vulcan Chemicals

For systems handling the 38 percent solution, storage tanks, piping and pumps will require a heated enclosure, or heat tracing and insulation. The 25 percent solution may not require any special protection except in cold climates.

The ideal production of 1.0 pound of chlorine dioxide requires 0.5 pounds of chlorine and 1.34 pounds of pure sodium chlorite. Chlorine gas is available as a nearly 100 percent pure chemical on a weight basis. Gas flow metering devices are typically limited to +/- 5 percent accuracy at full rated capacity. For example, a 100 pound per day flow tube would allow between 20 and 30 pounds of chlorine to flow if set at 25 pounds per day (i.e., 25 +/- 5 percent of maximum flow capacity). Sodium chlorite is supplied commercially as a pre-mixed aqueous solution of various strengths. The 25 percent solution is the most commonly used grade for potable water treatment.

Pure chlorine dioxide solutions (very dark amber and oily in appearance) are very dangerous and are likely to detonate if exposed to oxidizable materials or vapors, or even to bright lights. They are extremely uncommon except perhaps in very specific laboratory setup systems using concentrated sodium chlorite and concentrated acid mixtures. Such laboratory generation methods are not

recommended for the uninitiated laboratory analyst or operator. Inexperienced personnel should not mix strong acid and strong sodium chlorite solutions together unless they are familiar with the purgeable extraction methods for sodium chlorite and have a safely designed setup under a fume hood.

# 4.8 Summary

## 4.8.1 Advantages and Disadvantages of Chlorine Dioxide Use

The following list highlights selected advantages and disadvantages of using chlorine dioxide as a disinfection method for drinking water (Masschelein, 1992; DeMers and Renner, 1992, Gallagher et al., 1994). Because of the wide variation of system size, water quality, and dosages applied, some of these advantages and disadvantages may not apply to a particular system.

### Advantages

- Chlorine dioxide is more effective than chlorine and chloramines for inactivation of viruses, *Cryptosporidium*, and *Giardia*.
- Chlorine dioxide oxidizes iron, manganese, and sulfides.
- Chlorine dioxide may enhance the clarification process.
- Taste and odors resulting from algae and decaying vegetation, as well as phenolic compounds, are controlled by chlorine dioxide.
- Under proper generation conditions (i.e., no excess chlorine), halogen-substituted DBPs are not formed.
- Chlorine dioxide is easy to generate.
- Biocidal properties are not influenced by pH.
- Chlorine dioxide provides residuals.

### Disadvantages

- The chlorine dioxide process forms the specific byproducts chlorite and chlorate.
- Generator efficiency and optimization difficulty can cause excess chlorine to be fed at the application point, which can potentially form halogen-substitute DBPs.
- Costs associated with training, sampling, and laboratory testing for chlorite and chlorate are high.
- Equipment is typically rented, and the cost of the sodium chlorite is high.
- Measuring chlorine dioxide gas is explosive, so it must be generated on-site.
- Chlorine dioxide decomposes in sunlight.
- Chlorine dioxide must be made on-site.
- Can lead to production noxious odors in some systems.

## 4.8.2 Summary Table

Table 4-5 summarizes considerations and descriptions for chlorine dioxide use.

**Table 4-5. Summary for Chlorine Dioxide**

| Consideration | Description |
|---|---|
| Generation | Chlorine dioxide must be generated on-site. In most potable water applications, chlorine dioxide is generated as needed and directly educed or injected into a diluting stream. Generators are available that utilize sodium chlorite and a variety of feedstocks such as $Cl_2$ gas, sodium hypochlorite, and sulfuric or hydrochloric acid. Small samples of generated solutions, up to 1 percent (10 g/L) chlorine dioxide can be safely stored if the solution is protected from light, chilled (<5°C), and has no unventilated headspace. |
| Primary Uses | Chlorine dioxide is utilized as a primary or secondary disinfectant, for taste and odor control, TTHM/HAA reduction, Fe and Mn control, color removal, sulfide and phenol destruction, and Zebra mussel control. |
| Inactivation Efficiency | Chlorine dioxide rapidly inactivates most microorganisms over a wide pH range. It is more effective than chlorine (for pathogens other than viruses) and is not pH dependent between pH 5-10, but is less effective than ozone. |
| Byproducts Formation | When added to water, chlorine dioxide reacts with many organic and inorganic compounds. The reactions produce chlorite and chlorate as endproducts (compounds that are suspected of causing hemolytic anemia and other health effects). Chlorate ion is formed predominantly in downstream reactions between residual chlorite and free chlorine when used as the distribution system disinfectant. Chlorine dioxide does not produce THMs. The use of chlorine dioxide aids in reducing the formation of TTHMs and HAAs by oxidizing precursors, and by allowing the point of chlorination to be moved farther downstream in the plant after coagulation, sedimentation, and filtration have reduced the quantity of NOM. |
| Point of Application | In conventional treatment plants, chlorine dioxide used for oxidation is fed either in the raw water, in the sedimentation basins, or following sedimentation. To limit the oxidant demand, and therefore chlorine dioxide dose and the formation of chlorite, it is common to add chlorine dioxide following sedimentation. Concerns about possible taste and odor complaints have limited the use of chlorine dioxide to provide a disinfectant residual in the distribution system. Consequently, public water suppliers that are considering the use of chlorine dioxide for oxidation and primary disinfectant applications may want to consider chloramines for secondary disinfection. |
| Special Considerations | An oxidant demand study should be completed to determine an approximate chlorine dioxide dosage to obtain the required CT value as a disinfectant. In addition to the toxic effects of chlorine, chlorine dioxide gas is explosive at levels > 10% in air. The chlorine dioxide dosage cannot exceed 1.4 mg/L to limit the total combined concentration of ClO2, ClO2-, ClO3-, to a maximum of 1.0 mg/L. Under the proposed DBP regulations, the MRDL for chlorine dioxide is 0.8 mg/L and the MCL for chlorite is 1.0 mg/L. Regulations concerning the use of chlorine dioxide vary from state-to-state. |

## 4.9 References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# 5. POTASSIUM PERMANGANATE

Potassium permanganate ($KMnO_4$) is used primarily to control taste and odors, remove color, control biological growth in treatment plants, and remove iron and manganese. In a secondary role, potassium permanganate may be useful in controlling the formation of THMs and other DBPs by oxidizing precursors and reducing the demand for other disinfectants (Hazen and Sawyer, 1992). The mechanism of reduced DBPs may be as simple as moving the point of chlorine application further downstream in the treatment train using potassium permanganate to control taste and odors, color, algae, etc. instead of chlorine. Although potassium permanganate has many potential uses as an oxidant, it is a poor disinfectant.

## 5.1 Potassium Permanganate Chemistry

### 5.1.1 Oxidation Potential

Potassium permanganate is highly reactive under conditions found in the water industry. It will oxidize a wide variety of inorganic and organic substances. Potassium permanganate (Mn 7+) is reduced to manganese dioxide ($MnO_2$) (Mn 4+) which precipitates out of solution (Hazen and Sawyer, 1992). All reactions are exothermic. Under acidic conditions the oxidation half-reactions are (CRC, 1990):

$$MnO_4^- + 4H^+ + 3e^- \rightarrow MnO_2 + 2H_2O \qquad E^o = 1.68V$$

$$MnO_4^- + 8H^+ + 5e^- \rightarrow Mn^{2+} + 4H_2O \qquad E^o = 1.51V$$

Under alkaline conditions, the half-reaction is (CRC, 1990):

$$MnO_4^- + 2H_2O + 3e^- \rightarrow MnO_2 + 4OH^- \qquad E^o = 0.60V$$

Reaction rates for the oxidation of constituents found in natural waters are relatively fast and depend on temperature, pH, and dosage.

### 5.1.2 Ability To Form a Residual

It is not desirable to maintain a residual of $KMnO_4$ because of its tendency to give water a pink color.

## 5.2 Generation

Potassium permanganate is only supplied in dry form. A concentrated $KMnO_4$ solution (typically 1 to 4 percent) is generated on-site for water treatment applications; the solution is pink or purple in color. $KMnO_4$ has a bulk density of approximately 100 lb/ft$^3$ and its solubility in water is 6.4 g/mL at 20ºC.

Depending on the amount of permanganate required, these solutions can be made up in batch modes, using dissolver/storage tanks with mixers and a metering pump for small feed systems. Larger systems will include a dry chemical feeder, storage hopper and dust collector configured to automatically supply permanganate to the solution dissolver/storage tank.

$KMnO_4$ solution is made up of dry crystalline permanganate solids added to make-up water and then stirred to obtain the desired permanganate concentration. The cost of $KMnO_4$ ranges from $1.50 to $2.00 per pound (1997 costs), depending on the quantity ordered. Shipment containers are typically buckets or drums. Potassium permanganate is supplied in various grades. Pure $KMnO_4$ is non-hygroscopic but technical grades will absorb some moisture and will have a tendency to cake together. For systems using dry chemical feeders, a free-flowing grade is available that contains anti-caking additives (Hazen and Sawyer, 1992).

Potassium permanganate is a strong oxidizer and should be carefully handled when preparing the feed solution. No byproducts are generated from making the solution. However, this dark purple/black crystalline solid can cause serious eye injury, is a skin and inhalation irritant, and can be fatal if swallowed. As such, special handling procedures include the use of safety goggles and a face shield, an MSA™/NIOSH approved dust mask, and wearing impervious gloves, coveralls, and boots to minimize skin contact.

# 5.3 Primary Uses and Points of Application

Although potassium permanganate can inactivate various bacteria and viruses, it is not used as a primary or secondary disinfectant when applied at commonly used treatment levels. Potassium permanganate levels that may be required to obtain primary or secondary disinfection could be cost prohibitive. However, potassium permanganate is used in drinking water treatment to achieve a variety of other purposes including:

- Oxidation of iron and manganese;
- Oxidation of taste and odor compound;
- Control of nuisance organisms; and
- Control of DBP formation.

## 5.3.1 Primary Uses

### *5.3.1.1 Iron and Manganese Oxidation*

A primary use of permanganate is iron and manganese removal. Permanganate will oxidize iron and manganese to convert ferrous (2+) iron into the ferric (3+) state and 2+ manganese to the 4+ state. The oxidized forms will precipitate as ferric hydroxide and manganese hydroxide (AWWA, 1991). The precise chemical composition of the precipitate will depend on the nature of the water, temperature, and pH.

The classic reactions for the oxidation of iron and manganese are:

$$3Fe^{2+} + KMnO_4 + 7H_2O \rightarrow 3Fe(OH)_{3(s)} + MnO_{2(s)} + K^+ + 5H^+$$

$$3Mn^{2+} + 2KMnO_4 + 2H_2O \rightarrow 5MnO_{2(s)} + 2K^+ + 4H^+$$

These reactions show that alkalinity is consumed through acid production at the rate of 1.49 mg/L as $CaCO_3$ per mg/L of $Fe^{+2}$ and 1.21 mg/L as $CaCO_3$ per mg/L of $Mn^{+2}$ oxidized. This consumption of alkalinity should be considered when permanganate treatment is used along with alum coagulation, which also requires alkalinity to form precipitates.

The potassium permanganate dose required for oxidation is 0.94 mg/mg iron and 1.92 mg/mg manganese (Culp/Wesner/Culp, 1986). In practice, the actual amount of potassium permanganate used has been found to be less than that indicated by stoichiometry. It is thought that this is because of the catalytic influence of $MnO_2$ on the reactions (O'Connell, 1978). The oxidation time ranges from 5 to 10 minutes, provided that the pH is over 7.0 (Kawamura, 1991).

#### 5.3.1.2 Oxidation of Taste and Odor Compounds

Potassium permanganate is used to remove taste and odor causing compounds. Lalezary et al. (1986) used permanganate to treat earthy-musty smelling compounds in drinking water. Doses of potassium permanganate used to treat taste and odor causing compounds range from 0.25 to 20 mg/L.

#### 5.3.1.3 Control of Nuisance Organisms

##### Asiatic Clams

Cameron et al. (1989) investigated the effectiveness of potassium permanganate to control the Asiatic clam in both the juvenile and adult phases. The adult Asiatic clam was found to be much more resistant to permanganate than the juvenile form. Potassium permanganate doses used to control the juvenile Asiatic clam range from 1.1 to 4.8 mg/L.

##### Zebra Mussels

Klerks and Fraleigh (1991) evaluated the effectiveness of permanganate against adult zebra mussels. Continuous potassium permanganate dosing of 0.5 to 2.5 mg/L proved to be the most effective.

#### 5.3.1.4 DBP Control

It is anticipated that potassium permanganate may play a role in disinfection and DBP control strategies in water treatment. Potassium permanganate could be used to oxidize organic precursors at the head of the treatment plant minimizing the formation of byproducts at the downstream disinfection stage of the plant (Hazen and Sawyer, 1992). Test results from a study conducted at two water treatment plants in North Carolina (Section 5.5.1) showed that pretreatment with permanganate reduced chloroform formation; however, the reduction was small at doses typically used at water

treatment plants. The study also indicated that pre-oxidation with permanganate had no net effect on the chlorine demand of the water (Singer et al., 1980).

## 5.3.2 Points of Application

In conventional treatment plants, potassium permanganate solution is added to the raw water intake, at the rapid mix tank in conjunction with coagulants, or at clarifiers upstream of filters. In direct filtration plants, this oxidant is typically added at the raw water intake to increase the contact time upstream of the filter units (Montgomery, 1985). In all cases, potassium permanganate is added prior to filtration.

Potassium permanganate solution is typically pumped from the concentrated solution tank to the injection point. If the injection point is a pipeline, a standard injection nozzle protruding midway into the pipe section is used. Injection nozzles can also be used to supply the solution to mixing chambers and clarifiers. Permanganate is a reactive, fast-acting oxidizer and does not require special mixing equipment at the point of injection to be effective.

### *5.3.2.1 Impact on Other Treatment Processes*

The use of potassium permanganate has little impact on other treatment processes at the water treatment facility. See Section 5.7 for permanganate operational considerations.

# 5.4 Pathogen Inactivation and Disinfection Efficacy

Potassium permanganate is an oxidizing agent widely used throughout the water industry. While it is not considered a primary disinfectant, potassium permanganate has an effect on the development of a disinfection strategy by serving as an alternative to pre-chlorination or other oxidants at locations in a treatment plant where chemical oxidation is desired for control of color, taste and odor, and algae.

## 5.4.1 Inactivation Mechanisms

The primary mode of pathogen inactivation by potassium permanganate is direct oxidation of cell material or specific enzyme destruction (Webber and Posselt, 1972). In the same fashion, the permanganate ion ($MnO_4^-$) attacks a wide range of microorganisms such as bacteria, fungi, viruses, and algae.

Application of potassium permanganate results in the precipitation of manganese dioxide. This mechanism represents an additional method for the removal of microorganisms from potable water (Cleasby et al., 1964). In colloidal form, the manganese dioxide precipitant has an outer layer of exposed OH groups. These groups are capable of adsorbing charged species and particles in addition to neutral molecules (Posselt et al., 1967). As the precipitant is formed, microorganisms can be adsorbed into the colloids and settled.

## 5.4.2 Environmental Effects

Inactivation efficiency depends upon the permanganate concentration, contact time, temperature, pH, and presence of other oxidizable material. Several of the key parameters are discussed below.

### *5.4.2.1 pH*

Alkaline conditions enhance the capability of potassium permanganate to oxidize organic matter; however, the opposite is true for its disinfecting power. Typically, potassium permanganate is a better biocide under acidic conditions than under alkaline conditions (Cleasby et al., 1964 and Wagner, 1951). Results from a study conducted in 1964 indicated that permanganate generally was a more effective biocide for E. coli at lower pHs, exhibiting more than a 2-log removal at a pH of 5.9 and a water temperature of both 0 and 20°C (Cleasby et al., 1964). In fact, Cleasby found that pH is the major factor affecting disinfection effectiveness with potassium permanganate. As such, natural waters with pH values of 5.9 or less would be conducive to potassium permanganate disinfection, particularly as a substitute for prechlorination. Moreover a study conducted at the University of Arizona found that potassium permanganate will inactivate *Legionella pneumophila* more rapidly at pH 6.0 than at pH 8.0 (Yahya et al., 1990a).

These results are consistent with earlier results concerning the effects of pH on commercial antiseptic performance (Hazen and Sawyer, 1992). In general, based on the limited results from these studies, disinfection effectiveness of potassium permanganate increases with decreasing pH.

### *5.4.2.2 Temperature*

Higher temperatures slightly enhance bactericidal action of potassium permanganate. The results from a study conducted on polio virus showed that oxidation deactivation is enhanced by higher temperatures (Lund, 1963). These results are consistent with results obtained for *E. coli.* inactivation (Cleasby et al., 1964).

### *5.4.2.3 Dissolved Organics and Inorganics*

The presence of oxidizable organics or inorganics in the water reduces the disinfection effectiveness of this disinfectant because some of the applied potassium permanganate will be consumed in the oxidation of organics and inorganics. Permanganate oxidizes a wide variety of inorganic and organic substances in the pH range of 4 to 9. Under typical water conditions, iron and manganese are oxidized and precipitated and most contaminants that cause odors and tastes, such as phenols and algae, are readily degraded by permanganate (Hazen and Sawyer, 1992).

## 5.4.3 Use as a Disinfectant

A number of investigations have been performed to determine the relative capability of potassium permanganate as a disinfectant. The following sections contain a description of the disinfection efficiency of potassium permanganate in regards to bacteria, virus, and protozoa inactivation.

#### 5.4.3.1 Bacteria Inactivation

High dosage rates were required to accomplish complete inactivation of bacteria in three studies. Early research showed that a dose of 2.5 mg/L was required for complete inactivation of coliform bacteria (Le Strat, 1944). In this study, water from the Marne River was dosed with potassium permanganate at concentrations of 0 to 2.5 mg/L. Following mixing, the samples were placed in a darkened room for 2 hours at a constant temperature of 19.8°C.

Banerjea (1950) investigated the disinfectant ability of potassium permanganate on several waterborne pathogenic microorganisms. The investigation studied *Vibrio cholerae, Salm. typhi,* and *Bact. flexner*. The results indicated that doses of 20 mg/L and contact times of 24 hours were necessary to deactivate these pathogens; however, even under these conditions the complete absence of *Salm. typhi* or *Bact. flexner* was not assured, even at a potassium permanganate concentration that turned the water an objectionable pink color.

Results from a study conducted in 1976 at the Las Vegas Valley Water District/Southern Nevada System of Lake Mead water showed that complete removal of coliform bacteria were accomplished at doses of 1, 2, 3, 4, 5, and 6 mg/L (Hazen and Sawyer, 1992). Contact times of 30 minutes were provided with doses of 1 and 2 mg/L, and 10 minutes contact times were provided for higher dosages in this study.

#### 5.4.3.2 Virus Inactivation

Potassium permanganate has been proven effective against certain viruses. A dose of 50 mg/L of potassium permanganate and a contact time of 2 hours was required for inactivation of poliovirus (strain MVA) (Hazen and Sawyer, 1992). A "potassium" permanganate dose of 5.0 mg/L and a contact time of 33 minutes was needed for 1-log inactivation of type 1 poliovirus (Yahya et al., 1990b). Tests showed a significantly higher inactivation rate at 23°C than at 7°C; however, there was no significant difference in activation rates at pH 6.0 and pH 8.0.

Potassium permanganate doses from 0.5 to 5 mg/L were capable of obtaining at least a 2 log inactivation of the surrogate virus, MS-2 bacteriophage with *E. coli* as the host bacterium (Yahya et al., 1989). Results showed that at pH 6.0 and 8.0, a 2-log inactivation occurred after a contact time of at least 52 minutes and a residual of 0.5 mg/L. At a residual of 5.0 mg/L, approximately 7 and 13 minutes were required for 2-log inactivation at pHs of 8.0 and 6.0, respectively. These results contradict the previously cited studies that potassium permanganate becomes more effective as the pH decreases.

#### 5.4.3.3 Protozoa Inactivation

No information pertaining to protozoa inactivation by potassium permanganate is available in the literature. However, based on the other disinfectants discussed in this report, protozoa are significantly more resistant than viruses; therefore, it is likely that the dosages and contact times required for protozoa inactivation would be impractical.

#### 5.4.3.4 CT Curves

Table 5-1 shows CT values for the inactivation of bacteriophage MS-2. These data have been provided as an indication of the potential of potassium permanganate. These values are somewhat inconsistent and do not include a safety factor and should not be used to establish CT requirements.

**Table 5-1. Potassium Permanganate CT Values for 2-log Inactivation of MS-2 Bacteriophage**

| Residual (mg/L) | pH 6.0[1] (mg min / L) | pH 8.0[1] (mg min / L) |
|---|---|---|
| 0.5 | 27.4 (a) | 26.1 (a) |
| 1.5 | 32.0 (a) | 50.9 (b) |
| 2 | - | 53.5 (c) |
| 5 | 63.8 (a) | 35.5 (c) |

Source: USEPA, 1990.
Note: [1] Letters indicate different experimental conditions.

A 1990 study investigated CT values for *Legionella pneumophila* inactivation. CT values for 99 percent (2-log) inactivation of *Legionella pneumophila* at pH 6.0 were determined to be 42.7 mg min/L at a dose of 1.0 mg/L (contact time 42.7 minutes) and 41.0 mg min/L at a dose of 5.0 mg/L (contact time 8.2 minutes) (Yahya et al., 1990a).

## 5.5 Disinfection Byproduct Formation

No literature is available that specifically addressed DBPs when using potassium permanganate. However, several studies have been conducted with water treatment plants that have replaced the pre-chlorination process with potassium permanganate and relocated the point of chlorine addition for post-treatment disinfection. Pretreatment with permanganate in combination with post-treatment chlorination will typically result in lower DBP concentrations than would otherwise occur from traditional pre-chlorination (Ficek and Boll, 1980; and Singer et al., 1980). Under this approach, potassium permanganate serves as a substitute for chlorine to achieve oxidation and may also reduce the concentration of natural organic matter (NOM). However, systems should evaluate the impact on CT values before moving the point of chlorination. The following subsections summarize the outcomes of two studies.

### 5.5.1 Chapel-Hill and Durham, North Carolina Water Treatment Plants

An investigation was conducted at the Chapel-Hill and Durham Water Treatment Plants to evaluate the effects of potassium permanganate pretreatment on trihalomethane formation (Singer et al., 1980). The Chapel-Hill Water Treatment Plant uses pre-chlorination prior to the rapid mix tank. At the Durham Water Treatment Plant, chlorine is not added until after the sedimentation basin prior to the filtration. Both are surface water treatment plants, treating water with low concentrations of

alkalinity. Both sources of water are known to have high trihalomethane formation potentials (Young and Singer, 1979).

Raw water samples taken from Chapel-Hill were found to contain relatively high turbidities, ranging from 46 to 110 NTU and total organic carbon (TOC) concentrations ranging from 5.6 to 8.9 mg/L. The Durham samples were coagulated then allowed to settle, which resulted in better water quality than the Chapel-Hill samples. Following settling, this sample had a turbidity of 6.4 NTU and a TOC of 2.9 mg/L. Sulfuric acid and sodium hydroxide were used to adjust the sample pH to either 6.5 or 10.3. These pH values were selected because they encompass the pH range typically found in surface water coagulation-filtration and lime-softening treatment plants.

Potassium permanganate doses of 2 and 5 mg/L were found to be totally consumed within 1 and 4 hours, respectively, by the Chapel-Hill samples. At doses of 2 and 5 mg/L, the potassium permanganate demand of the Durham samples after 4 hours were approximately 1.3 and 1.8 mg/L, respectively.

This difference in permanganate demands between the Chapel-Hill and Durham samples may be attributed to the water quality of the samples, in particular the TOC concentrations. TOC measurements before and after the application of permanganate were approximately equal; however, it is likely that the TOC after disinfection was at a higher oxidation state. Results of this study also showed that permanganate is more reactive as an oxidant at higher pH values.

Despite the high degree of permanganate consumption, the reaction of permanganate appears to have relatively little effect on chlorine demands. For example, consumption of 6 mg/L of permanganate resulted in a chlorine demand reduction of approximately 1 mg/L. This observation suggests that permanganate reacts with water impurities in a different manner, or at different sites, than chlorine. One other possible explanation is that permanganate oxidizes certain organic substances, thereby eliminating their chlorine demand and only partially oxidizing other organic substances making them more reactive to chlorine.

Both the Chapel-Hill and Durham samples were tested for their chloroform formation potential. This measurement is based on the amount of chloroform produced after seven days. The potential of the Durham sample was reduced by 30 and 40 percent at pH 6.5 and 10.3, respectively, as a result of the application of 10 mg/L of potassium permanganate for a period of 2 hours. Similar results were obtained for the Chapel-Hill samples; however, the results at pH 6.5 did not show a reduction in chloroform formation potential at low doses.

Two experiments were conducted on Chapel-Hill raw water to further explore the effects of low doses of permanganate. The results indicated that permanganate has no effect on chloroform production at doses up to 1 mg/L. At higher doses, chloroform formation potentials were reduced.

In summary, the key results obtained from the studies conducted at the Chapel-Hill and Durham Water treatment plants were:

- The reactivity of permanganate is a function of pH, permanganate dose, and raw water quality.
- Permanganate reduces chloroform formation potentials. The reduction in the chloroform formation potential is proportional to the amount of permanganate available after the initial demand is overcome. Doses up to 1 mg/L were found to have no effect on chloroform formation potentials.
- At pretreatment doses typically employed at water treatment plants, the effect of permanganate on the overall chloroform production is relatively small. If permanganate is to be used specifically to reduce trihalomethane formation, larger doses will be required. However, one advantage for using permanganate for pretreatment is that the point of application of chlorine can be shifted downstream of the sedimentation basins. This is likely to result in fewer trihalomethane compounds.

### 5.5.2 American Water Works Association Research Foundation TTHM Study

Another investigation examined the impacts of potassium permanganate addition on byproduct formation at four water treatment plants (Ficek and Boll, 1980). All were conventional plants using pre-chlorination in the treatment process. Plant design capacities ranged from 4.5 to 15 mgd. Process modifications were made at each plant to replace the pre-chlorination facilities with oxidation facilities for potassium permanganate addition. After the modifications were complete, an AWWARF research team conducted a study to determine the impact of potassium permanganate addition on total trihalomethane (TTHM) concentrations (George et al., 1990).

Prior to switching from pre-chlorination to pre-oxidation with potassium permanganate, average daily TTHM concentrations at all four plants were between 79 and 99 μg/L. The average TTHM concentration for all four plants was 92 μg/L. Following the conversion to potassium permanganate, three of the four plants experienced greater than 30 percent reduction in TTHM concentrations. In addition to TTHM reduction, potassium permanganate was found to oxidize taste and odor causing compounds, iron and manganese, organic and inorganic matter, and reduce algal growth. Results from the study also showed that the simultaneous application of potassium permanganate and chlorine can increase THM formation.

## 5.6 Status of Analytical Methods

The atomic adsorption spectrophotometry method for the measurement of manganese is the preferred method for measuring permanganate concentrations. Two colorimetric methods, persulfate and periodate are also available (Standard Methods, 1995).

## 5.7 Operational Considerations

In utilizing potassium permanganate in water treatment, caution should be taken to prevent overdosing, in which case, excess manganese will pass through the treatment plant. Proper dosing

should be maintained to ensure that all of the permanganate is reduced (i.e., forming $MnO_2$ solids) and removed from the plant upstream of, or within, the filters. If residual manganese is reduced downstream of the filters, the resulting solids can turn the finished water a brown/black color and precipitate in the homes of consumers on heat exchange surfaces such as hot water heaters and dishwashers.

Use of potassium permanganate can also be a source of manganese in the finished water, which is regulated in drinking water with a secondary maximum contaminant level of 0.05 mg/L. Under reducing conditions, the $MnO_2$ solids accumulated in filter backwash water and settling basins can be reduced to soluble $Mn^{2+}$ and pass through the filters thereby remaining in the finished water.

Also, under these conditions, soluble $Mn^{2+}$ in return water from settling basin dewatering facilities and filter backwash water recycled to the head of the plant are potential sources of manganese that will have to be treated and/or controlled to minimize finished water manganese levels (Singer, 1991).

Overdosing of permanganate in conventional plants is generally corrected by settling the excess $MnO_2$ solids in the settling basin. Removal of the excess permanganate can be monitored qualitatively by observing the disappearance of the pink color characteristic of permanganate. In plants that do not utilize flocculation and sedimentation processes permanganate dosing should be closely monitored (Montgomery, 1985).

In general, potassium permanganate does not interfere with other treatment processes or plant conditions. Permanganate can be added downstream of, or concurrently with, coagulant and filter polymer aids. Powdered activated carbon (PAC) and permanganate should not be added concurrently. PAC should be added downstream of permanganate because it may consume permanganate, rendering it unavailable for the oxidation of target organics. (Montgomery, 1985).

The space requirements for permanganate feed equipment vary depending on the type and size of feed system. Dry feed systems require about half the floor area of batch systems because batch systems typically have two dissolving tanks for redundancy. However, the head space requirements are greater for dry feed systems where the storage hopper and dust collector are stacked on top of the dry feeder (Kawamura, 1991). On-site storage of potassium permanganate also warrants some consideration. Per OSHA requirements, oxidants such as permanganate should be stored separate from organic chemicals such as polymers and activated carbon.

## 5.8 Summary

### 5.8.1 Advantages and Disadvantages of Potassium Permanganate Use

The following list highlights selected advantages and disadvantages of using potassium permanganate as a disinfection method for drinking water. Because of the wide variation of system

size, water quality, and dosages applied, some of these advantages and disadvantages may not apply to a particular system.

### Advantages

- Potassium permanganate oxidizes iron and manganese.
- Potassium permanganate oxidizes odor and taste-causing compounds.
- Potassium permanganate is easy to transport, store, and apply.
- Potassium permanganate is useful in controlling the formation of THMs and other DBPs.
- Potassium permanganate controls nuisance organisms.
- The use of potassium permanganate has little impact on other treatment processes at the water treatment facility.
- Potassium permanganate has been proven effective against certain viruses.

### Disadvantages

- Long contact time is required.
- Potassium permanganate has a tendency to give water a pink color.
- Potassium permanganate is toxic and irritating to skin and mucous membranes.
- No byproducts are generated when preparing the feed solution, however this dark purple/black crystalline solid can cause serious eye injury, is a skin and inhalation irritant, and can be fatal if swallowed. Over-dosing is dangerous and may cause health problems such as chemical jaundice and drop in blood pressure.

## 5.8.2 Summary Table

More research is needed regarding the disinfection properties and oxidation byproducts of permanganate in water treatment. Also, a CT credit needs to be assigned to permanganate if it is to be utilized as a disinfectant. However, given that alternative oxidants, such as ozone and chlorine dioxide, demonstrate much greater efficacy in microbial control, permanganate is not likely to be utilized as a primary oxidant for precursor control. Table 5-2 summarizes the information presented in this chapter regarding the use of potassium permanganate in the drinking water treatment process.

**Table 5-2. Summary of Potassium Permanganate Use**

| Consideration | Description |
|---|---|
| Generation | Product supplied in dry form in buckets, drums, and bulk. On-site generation of solution is required using chemical mixing and feed equipment. |
| Primary uses | Control of odor and taste, remove color, control biological growth, and remove iron and manganese. |
| Inactivation efficiency | Not a good disinfectant. Can serve better as an alternative to chlorine or other disinfectants where chemical oxidation is desired. |
| Byproduct formation | No literature was found that specifically addressed DBP formation from potassium permanganate oxidation. Pretreatment with permanganate in combination with post-treatment chlorination will typically result in lower DBP concentrations than would otherwise occur from traditional pre-chlorination. |
| Limitations | Not a good disinfectant; primarily used for pretreatment to minimize chlorine usage and byproduct formation. |
| Points of application | Conventional Treatment: raw water addition, rapid mix tank in conjunction with coagulants, clarifiers upstream of filters. Direct Filtration: raw water intake. In all cases permanganate should be added upstream of filters. |
| Special considerations | Caution should be taken to prevent overdosing. More research is needed to determine disinfection properties and oxidation byproducts. |

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**5. POTASSIUM PERMANGANATE ..... 5-1**

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# 6. CHLORAMINES

The disinfectant potential of chlorine-ammonia compounds or chloramines was identified in the early 1900s. The potential use of chloramines was considered after observing that disinfection by chlorine occurred in two distinct phases. During the initial phase, chlorine reducing compounds (i.e., demand) cause the rapid disappearance of free available chlorine. However, when ammonia was present bactericidal action was observed to continue [even though free chlorine residual was dissipated]. The subsequent disinfection phase occurs by the action of the inorganic chloramines.

## 6.1 Chloramines Chemistry

Chloramines are formed by the reaction of ammonia with aqueous chlorine (i.e., HOCl). Initially, chloramines were used for taste and odor control. However, it was soon recognized that chloramines were more stable than free chlorine in the distribution system and consequently were found to be effective for controlling bacterial regrowth. As a result, chloramines were used regularly during the 1930s and 1940s for disinfection. Due to an ammonia shortage during World War II, however, the popularity of chloramination declined. Concern during the past two decades over chlorinated organics (e.g., THM and HAA formation) in water treatment and distribution systems, increased interest in chloramines because they form very few disinfection byproducts (DBPs).

### 6.1.1 Equilibrium, Kinetic, and Physiochemical Properties

Chloramines are formed from the reaction of chlorine and ammonia. The mixture that results may contain monochloramine ($NH_2Cl$), dichloramine ($NHCl_2$), or nitrogen trichloride ($NCl_3$). When chlorine is dispersed in water, a rapid hydrolysis occurs according to the following reaction:

$$Cl_2 + H_2O \rightarrow HOCl + H^+ + Cl^-$$

The equilibrium constant ($K_{eq}$) at 25°C is $3.94 \times 10^4$ $M^{-1}$ for this reaction. In dilute solutions at pH greater than 3, the forward reaction is essentially complete. Hypochlorous acid (HOCl) is a weak acid that dissociates as follows:

$$HOCl \Leftrightarrow OCl^- + H^+ \qquad pK_a = 7.6$$

Relative proportions of HOCl and $OCl^-$ are dependent upon pH. Both of the chlorine species in the above reaction are powerful oxidants, capable of reacting with many substances present in water. In aqueous solutions with pH 7.0 to 8.5, HOCl reacts rapidly with ammonia to form inorganic chloramines in a series of competing reactions (White, 1992). The simplified stoichiometry of chlorine-ammonia reactions are as follows:

$$NH_3 + HOCl \rightarrow NH_2Cl + H_2O \qquad \text{(monochloramine)}$$

$$NH_2Cl + HOCl \rightarrow NHCl_2 + H_2O \qquad \text{(dichloramine)}$$

$$NHCl_2 + HOCl \rightarrow NCl_3 + H_2O \qquad \text{(nitrogen trichloride)}$$

These competing reactions, and several others, are primarily dependent on pH and controlled to a large extent by the chlorine:ammonia nitrogen ($Cl_2$:N) ratio. Temperature and contact time also play a role. Figure 6-1shows the typical relationships between the chloramine species at various $Cl_2$:N ratios for pHs ranging from 6.5 to 8.5. This figure shows that monochloramine is predominately formed when the applied $Cl_2$:N ratio is less than 5:1 by weight. As the applied $Cl_2$:N ratio increases from 5:1 to 7.6:1, breakpoint reaction occurs, reducing the residual chlorine level to a minimum. Breakpoint chlorination results in the formation of nitrogen gas, nitrate, and nitrogen chloride. At $Cl_2$:N ratios above 7.6:1, free chlorine and nitrogen trichloride are present. Figure 6-2 shows the relationship between chloramine species as the pH changes (Palin, 1950). The Figure shows that dichloramine becomes a dominant species at low pH.

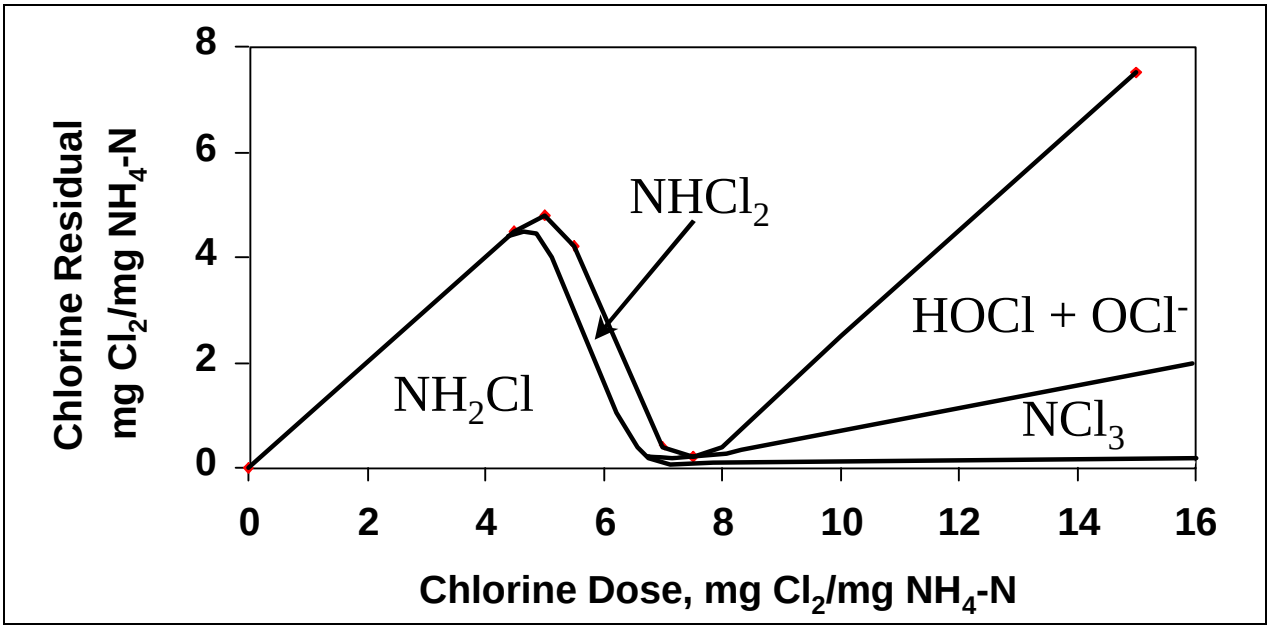
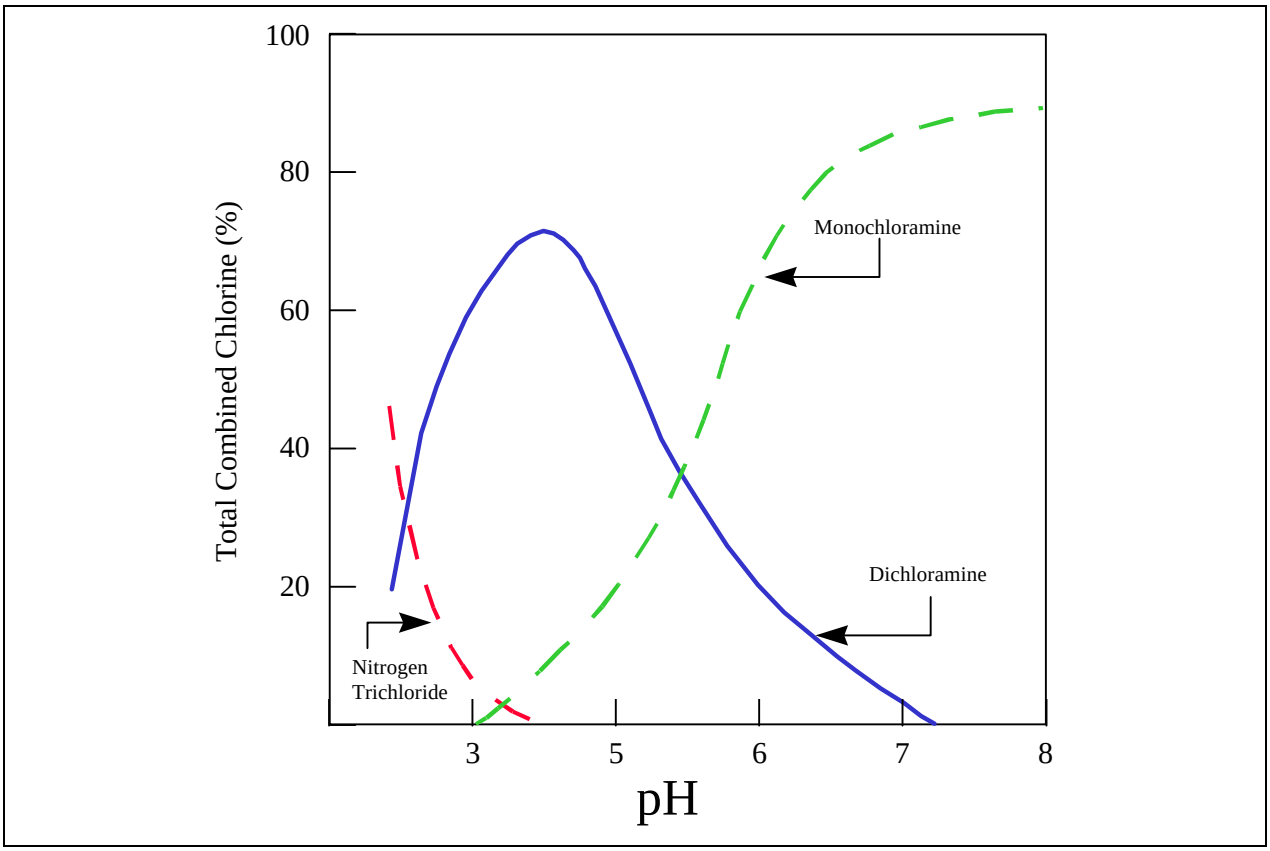



**Figure 6-1. Theoretical Breakpoint Curve**

To avoid breakpoint reactions, utilities should maintain a $Cl_2$:N ratio between 3 and 5 by weight. A ratio of 6 is actually optimum for disinfection, but it is difficult to maintain a stable operation at that point in the breakthrough curve. Therefore, a $Cl_2$:N ratio of 4 is typically accepted as optimal for chloramination.

Furthermore, over a period of a day or so, without any modification of pH or $Cl_2$:N ratio, monochloramine will degrade slowly to dichloramine to a ratio of 43 percent $NH_2Cl$ to 57 percent $NHCl_2$. Dichloramine is relatively unstable in the presence of HOCl; therefore, pure solutions of this form of monochloramine are difficult to generate and maintain.




Source: Palin, 1950.

**Figure 6-2. Distribution Diagram for Chloramine Species with pH**

## 6.2 Generation

Chloramines are formed by the reaction of hypochlorous acid and ammonia according to the equations described in Section 6.1. Table 6-1 summarizes the theoretical doses of chlorine and ammonia based on these formulas. Monochloramine is the preferred chloramine species for use in disinfecting drinking water because of taste and odor problems associated with dichloramine and nitrogen trichloride. To ensure that these compounds are not formed, common practice was to limit the chlorine to ammonia ratio to 3:1. However, because of problems such as nitrification and biofilm growth, which can be caused by excess ammonia, current practice is to use a $Cl_2$:N ratio in the range of 3:1 to 5:1, with a typical value of 4:1.

**Table 6-1. Chlorine Dose Required for $NH_3$ - $Cl_2$ Reaction**

| Reaction | mg $Cl_2$/mg $NH_3$ |
|---|---|
| Monochloramine ($NH_2Cl$) | 4.2 |
| Dichloramine ($NHCl_2$) | 8.4 |
| Nitrogen Trichloride ($NCl_3$) | 12.5 |
| Nitrogen ($N_2$) | 6.3 |
| Nitrate ($NO_3$) | 16.7 |
| Free residual reaction | 9 |

Source: AWWA and ASCE, 1990.

The rate of reaction of monochloramine formation is sensitive to pH. Table 6-2 shows the calculated reaction times for monochloromine formation at 25°C, and at a chlorine:amonia ratio of 3:1 (White, 1992).

**Table 6-2. Time to 99 Percent Conversion of Chlorine to Monochloramine**

| pH | Time (seconds) |
|---|---|
| 2 | 421 |
| 4 | 147 |
| 7 | 0.2 |
| 8.3 | 0.069 |
| 12 | 33.2 |

## 6.2.1 Chlorine Feed Facilities

Table 6-3 summarizes commonly used methods of chlorine addition, including their safety precautions and costs.

**Table 6-3. Methods of Chlorine Addition**

| Method | Description | Safety precautions | Costs |
|---|---|---|---|
| Gaseous chlorine | Gas delivered in containers ranging in size from 150 lb cylinders to 90 ton rail cars. One ton cylinders are commonly used. Feed equipment consists of solution water pump/ejector to create vacuum and automatic orifice control to meter the gas. Gas can be drawn directly from storage container or be generated by an evaporator from liquid withdrawn from the container. A schematic of gaseous chlorine feed system is shown in Figure 6-3. | Gaseous chlorine is classified by the Uniform Fire Code as an oxidizing, highly toxic, compressed gas. New gaseous chlorine facilities should be designed with enclosures and air scrubbers to capture and neutralize any gas that leaks. Risk management prevention plans should be prepared. Personnel safety equipment and training should be provided for operators. | The cost per pound of liquid chlorine is in the range of $0.08 to $0.20 per pound depending on the quantity purchased. |
| Sodium hypochlorite | Sodium hypochlorite can be purchased bulk in quantities ranging from 55 gal drums to 4,500 gal truck loads. Bulk loads can be stored in fiberglass or plastic tanks. Solution is fed directly into the process stream. A schematic of typical hypochlorite feed system is shown in Figure 6-4. | Hypochlorite solution is toxic and classified as hazardous. Storage facilities should be designed with secondary containment. | Typical chemical cost is $0.60 to $1.00 per pound $Cl_2$. |

## 6.2.2 Ammonia Feed Facilities

Ammonia feed facilities can be located on-site at the water treatment plant or at remote locations in the distribution system (Dennis et al., 1991). Most ammonia feed facilities use either gaseous (anhydrous ammonia) or liquid (aqueous) ammonia. Though anhydrous ammonia is a gas at ambient temperature and pressure, it is commonly stored and transported as a liquid in pressure vessels. In this phase, ammonia is highly soluble in water. Storage facilities and handling equipment should be kept dry (Dennis et al., 1991).

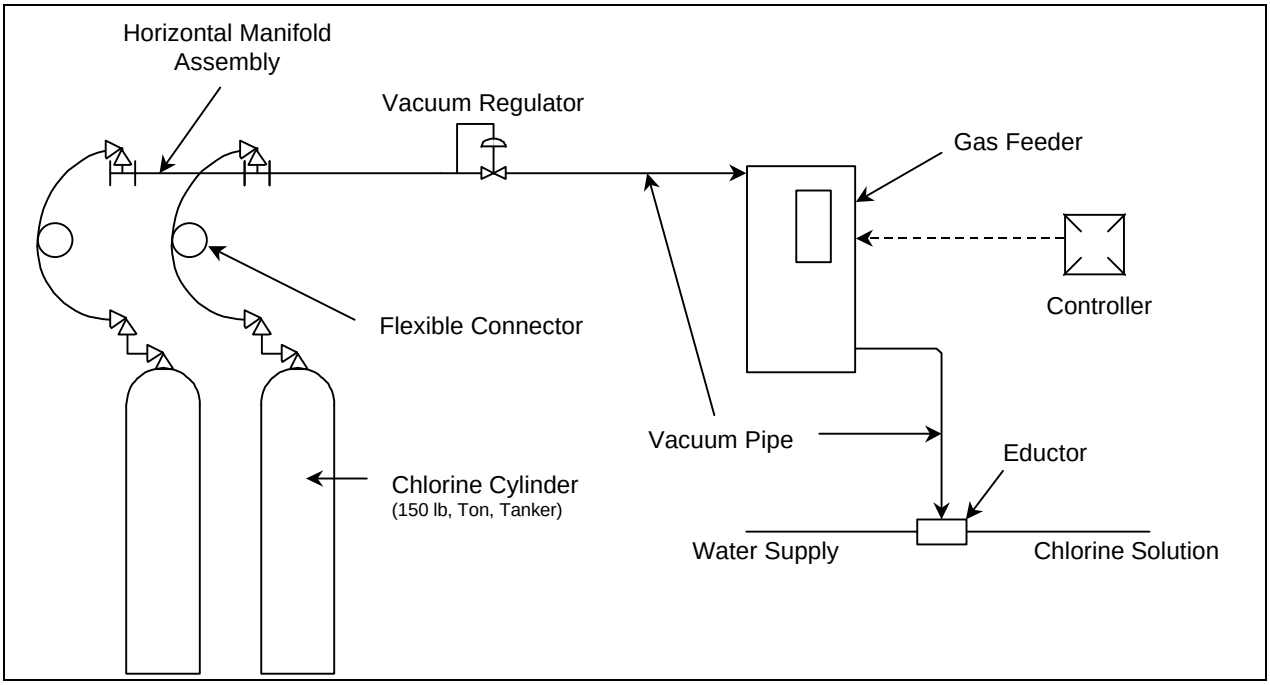



**Figure 6-3. Gaseous Chlorine Feed System**

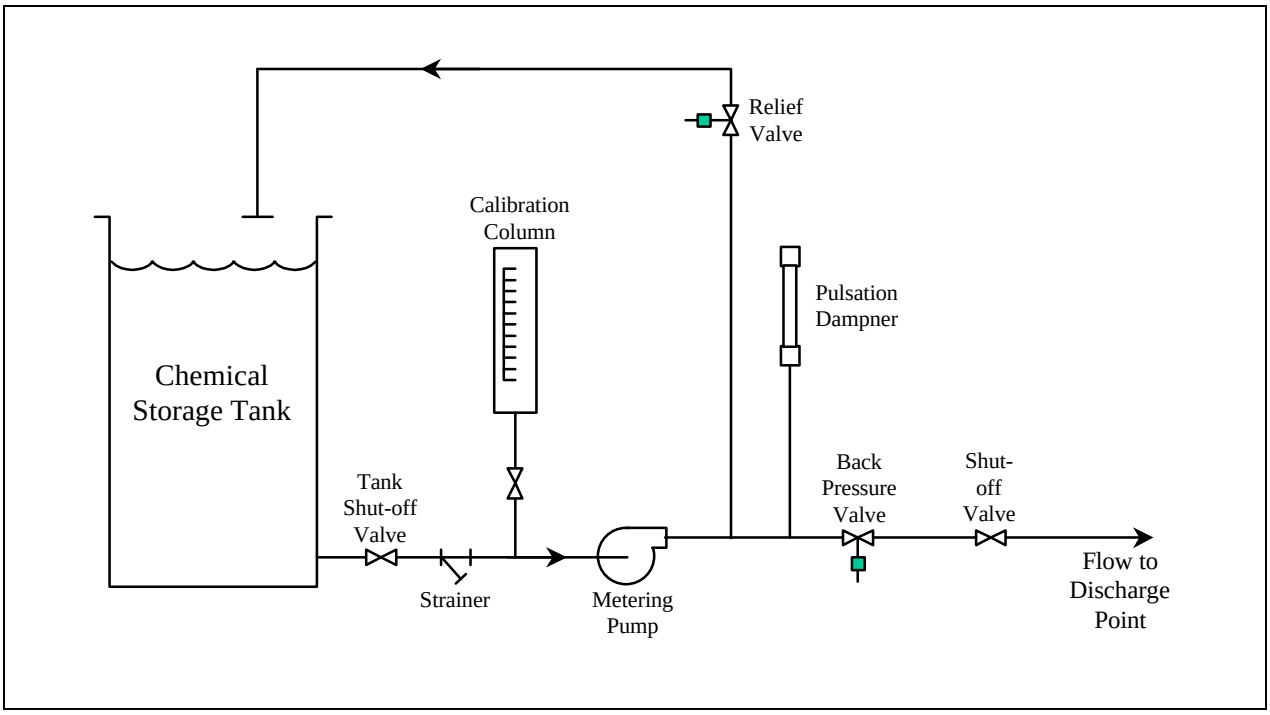



**Figure 6-4. Hypochlorite Feed System**

### *6.2.2.1 Anhydrous Ammonia*

Anhydrous ammonia is stored in portable cylinders or stationary tanks. Portable cylinders are similar to chlorine cylinders and are available in 100, 150, and 800 lb sizes (Dennis et al., 1991). The cylinders are rated for a minimum service pressure of 480 psi. Stationary tanks are typically 1,000 gallon vessels that can be used on-site. These tanks are refilled by tanker trailers. The storage tanks can be located indoors or outdoors. Since each tank has a minimum working pressure of 250 psi (valves and fittings on the tanks are rated for 300 psi), a tank stored outdoors should have protection from extreme temperatures (greater than 125°F and less than 28°F) (Dennis et al., 1991). In warmer climates, an outdoor tank should be painted white and protected from sunlight. In colder climates, the tank should be wrapped with heat tape to prevent impairment of the ammonia vaporization.

Anhydrous ammonia is applied using an ammoniator. An ammoniator is a self-contained modular unit with a pressure reducing valve, gas flow meter, feed rate control valve, and miscellaneous piping for controlling the flow of ammonia. Automatic paced ammoniators are available. An evaporator is used when large quantities of ammonia gas are needed. An anti-siphon valve or check valve should be used to prevent water from entering the ammoniator.

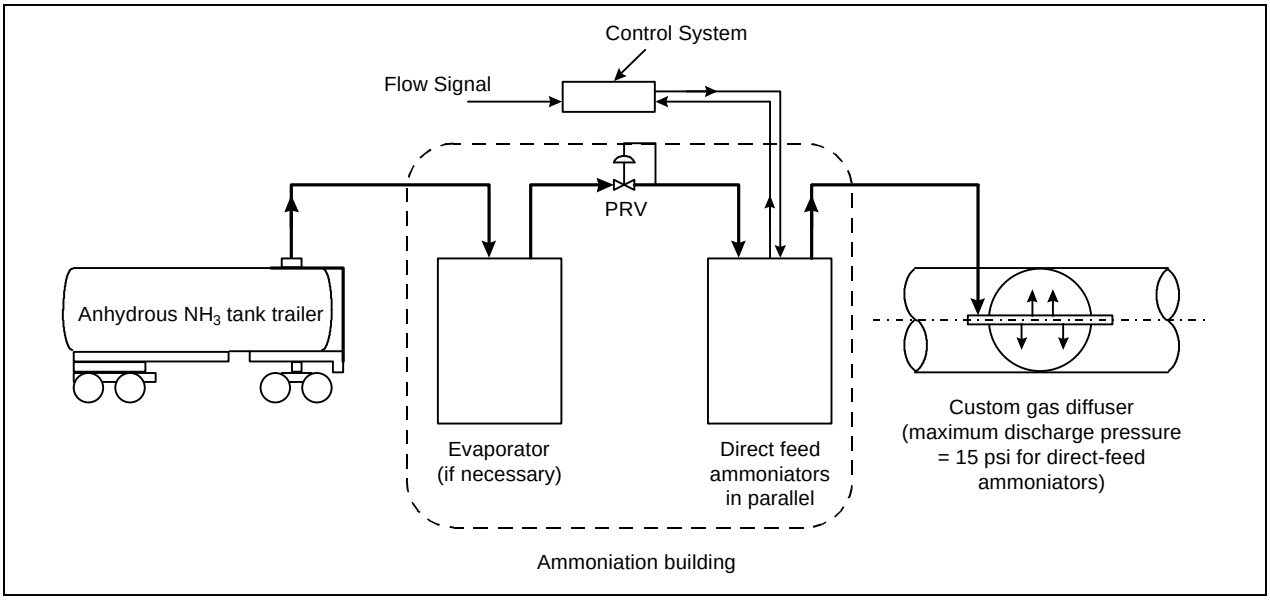
Anhydrous ammonia is usually applied by direct feed or solution feed. The direct feed method is typically used when the process stream has a low pressure and the ammonia feed rate is less than 1,000 lb per day (i.e., maximum rated feed capacity). Ammonia is drawn from the storage tank under high pressure (e.g., 200 psi), and injected directly into the process stream at a low pressure of 15 psi. The tank pressure is first reduced by a pressure reducing valve to approximately 40 psi, and

then by another pressure reducing valve in the ammoniator. Typical application points are at open channels and basin facilities. Figure 6-5 is a schematic of a direct anhydrous ammonia feed system.

The solution feed method is typically used where direct feed systems are not adequate (e.g., ammonia feed rate is greater than 1000 lb/day or where the process stream pressure is high) (Dennis et al., 1991). This type of application is similar to the chlorine vacuum feed system. The supply tank pressure is reduced by a pressure reducing valve to create a vacuum. An eductor is used to withdraw ammonia from the ammoniator where the ammonia is dissolved into a side water stream and pumped into the process stream. Solution feed ammoniators are available up to 4,000 lb/day capacities and can operate at discharge pressures up to 100 psi (Dennis et al., 1991). Softened water (i.e., hardness less than 29 mg/L as $CaCO_3$) is required for the carrier stream. Otherwise, the ammonia addition will precipitate scale that may plug the eductor and application point. Figure 6-6 shows a schematic of a solution feed system.

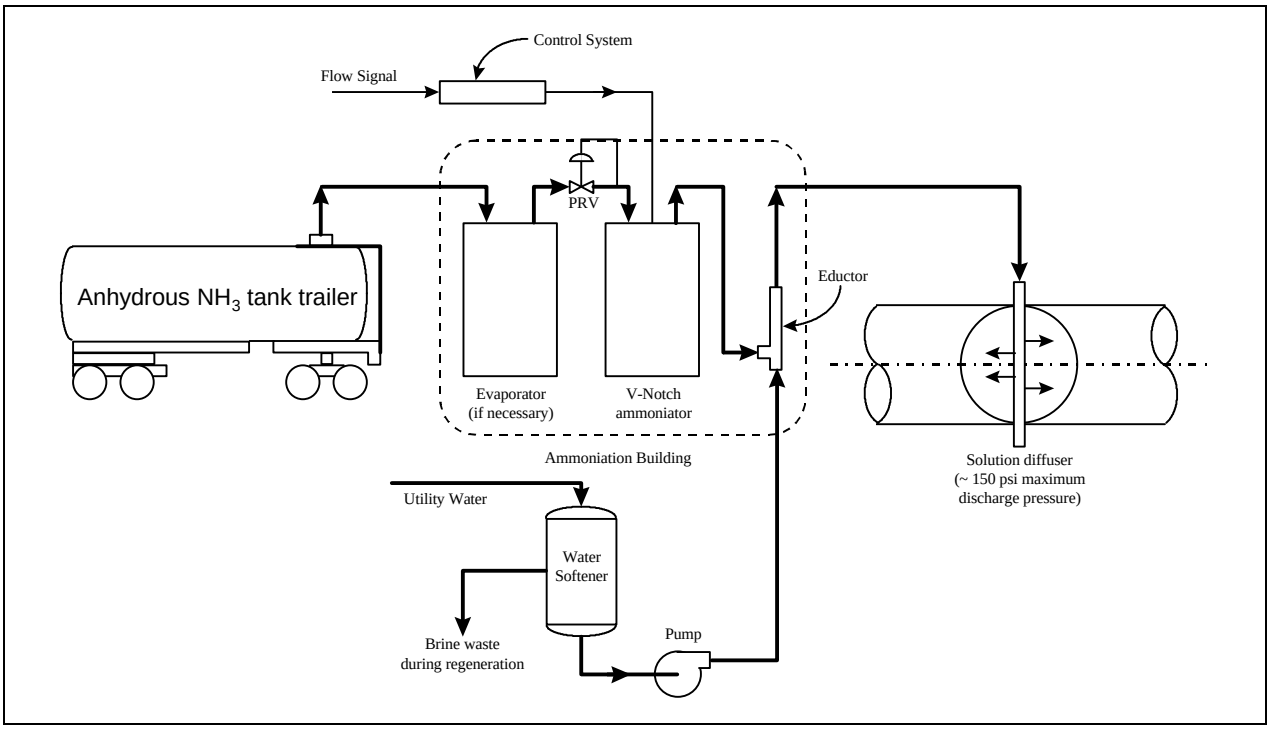
### 6.2.2.2 Aqueous Ammonia

Aqueous ammonia is produced by dissolving anhydrous ammonia into deionized or softened water. This form of ammonia is shipped in cargo trucks or polyethylene lined steel drums. Plastic drums are not recommended since they tend to lose their shape under the slight pressure exerted by the aqueous ammonia. Aqueous ammonia is stored in low pressure tanks, typically steel or fiberglass. Since excessive temperatures will cause ammonia gas to vaporize, each storage tank should be equipped with a water trap or ammonia scrubber to keep vapors from escaping to the atmosphere.




Source: Montgomery, 1985.

**Figure 6-5. Anhydrous Ammonia Direct Feed System**

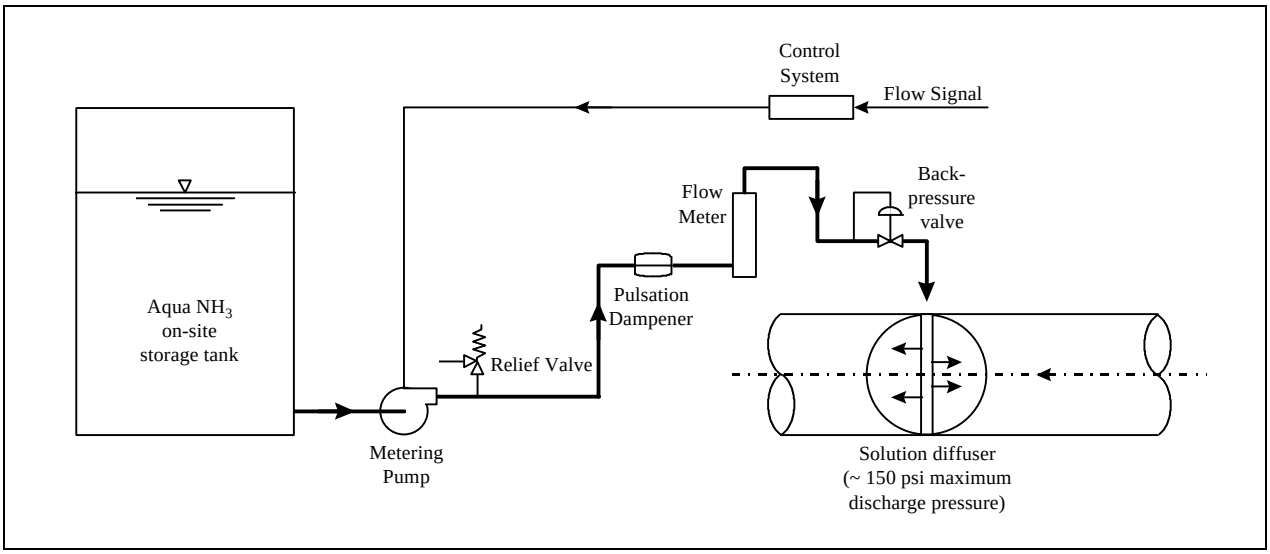



Source: Montgomery, 1985.

**Figure 6-6. Anhydrous Ammonia Solution Feed System**

Aqueous ammonia feed systems are similar to other liquid chemical feed systems. They require a storage tank, chemical metering pump, relief valve, pulsation dampener, flow meter, and backpressure valve. Typically, the feed pumps are positive displacement or progressive cavity type metering pumps. The feed pumps should be placed fairly close to the storage tank to minimize chances of ammonia vaporization in the piping (Dennis et al., 1991). The pump should be designed to compensate for changes in ambient temperatures, different aqueous ammonia solutions, and changes in the chlorine-to-ammonia ratio (Skadsen, 1993). When aqueous ammonia is applied to water, complete mixing is required for the ammonia to react with chlorine in the water to reduce the formation of dichloramine and nitrogen trichlorine. Figure 6-7 shows a schematic of an aqua ammonia feed system.

Metropolitan Water District of Southern California (MWDSC) uses aqueous ammonia at its chloramination facility. Ammonia is stored in unlined tanks and pumped to the ammoniator with progressive cavity pumps. During startup of its aqueous ammonia feed system, MWDSC experienced complete pump failures. Based on MWDSC's experience, EPDM rotors with adequate quality chromed finish stators are recommended for progressive cavity pumps. A mechanical seal is also recommended instead of a packing box to reduce the possibility of ammonia leaks (Skadsen, 1993). MWDSC also later installed special blow-offs and strainers in the feed pump suction line to reduce plugging at the magnetic flow meters. The pump problems prior to startup led MWDSC to install an alternative, redundant ammonia feed system. A pressurized system was designed to feed aqueous ammonia by pressurizing the ammonia tanks and by-passing the pump.




Source: Montgomery, 1985.

**Figure 6-7. Aqua Ammonia Feed System**

A 5.5 gpm flexible impeller centrifugal pump with a recirculation loop back to the storage tank regulates the back pressure on the by-passed feed pump. This alternative, redundant method proved to be reliable and economical. In addition, it provided a stable feed rate and required little maintenance (Skadsen, 1993).

### *6.2.2.3 Piping and Valving*

For anhydrous ammonia, the typical piping materials for both direct and solution feed systems are stainless steel, PVC, and black iron (Dennis et al., 1991). Stainless steel or black iron pipe is used in the high pressure (i.e., greater than 15 psi) portions of the feed system. PVC pipe is used only in the low pressure portion of the feed system, after the ammoniators.

For aqueous ammonia, PVC piping should be used due to the corrosive nature of aqueous ammonia (Dennis et al., 1991).

### *6.2.2.4 Safety Provisions for Chloramine Generation Facilities*

A chloramination facility should include some safety provisions to prevent the formation of nitrogen trichloride and the vaporization of ammonia at ambient temperatures. The possible formation of nitrogen trichloride at a chloramination facility should be considered when selecting sites for the ammonia and chlorine storage facilities.

Dennis et al. (1991) provides detailed information about safety provisions for chloramine facilities. Chlorine gas and ammonia gas should never be stored in the same room. The ammonia gas application points should be located at least 5 feet away from chlorine feed solution lines. Anhydrous ammonia is lighter than air, so any leaking vapor will rise quickly. Under pressure, anhydrous ammonia is a liquid. Great amounts of heat are absorbed when the pressurized liquid reverts to a gas.

If the storage tanks and/or chemical feed equipment are installed indoors, ventilation and vapor detection devices should be located at high points in the room. The ventilation rates will vary depending on the appropriate regulatory agency's requirements. Typically, a minimum of 6 room volume changes per minute is recommended.

Ammonia gas storage tanks should be protected from direct sunlight or direct sources of heat (i.e., greater than 125°F) to avoid pressure increases in the tank (Dennis et al., 1991). Otherwise, ammonia gas may be released into the atmosphere through the pressure relief valves. In warm regions, outdoor tanks should be covered with a shelter or outfitted with a temperature control sprinkler system. Where fugitive emissions of ammonia are a concern, fume control may be required. If the accidental release from a storage container is a concern, an emergency scrubber system similar to a chlorine gas scrubber system should be considered.

# 6.3 Primary Uses and Points of Application

Monochloramine is used in drinking water treatment for disinfection and nuisance organism control. Points of application are based on treatment objectives and contact time disinfection requirements.

## 6.3.1 Primary Uses

### *6.3.1.1 Disinfection*

The primary use of monochloramine in water systems is as a secondary disinfectant for maintaining a residual in the distribution system. Chloramines are a good choice for secondary disinfectant because of the following potential benefits:

- Chloramines are not as reactive with organics as free chlorine in forming THMs.
- The monochloramine residual is more stable and longer lasting than free chlorine or chlorine dioxide, providing better protection against bacterial regrowth in systems with large storage tanks and dead-end water mains.
- The monochloramine residual has been shown to be more effective in controlling biofilms because of its superior ability to penetrate the biofilm. Controlling biofilms also tends to reduce coliform concentrations and biofilm induced corrosion.
- Because chloramines do not tend to react with organic compounds, many systems will experience less incidence of taste and odor complaints when using chloramines.

Water systems in Indiana and Virginia found that conversion from free chlorine to monochloramine as the secondary disinfectant significantly reduced coliform concentrations in the distribution system (Norton and LeChevallier, 1997).

The normal dosage range for monochloramine is in the range of 1.0 to 4.0 mg/L. The minimum residual of monochloramine in the distribution system is typically regulated at 0.5 mg/L (Texas

Natural Resource Conservation Commission). For prevention of nitrification in a distribution system, a minimum monochloramine dosage of 2.0 mg/L is recommended (Norton and LeChevallier, 1997).

#### 6.3.1.2 Nuisance Organism Control

Cameron et al. (1989a) investigated the effectiveness of monochloramine to control the Asiatic clam in both the juvenile and adult phases. The adult Asiatic clam was found to be much more resistant to monochloramine than the juvenile form. Monochloramine was found to be the most effective for controlling the juvenile Asiatic clam in terms of $LT_{50}$ (time required for 50 percent mortality). Monochloramine doses used to control the juvenile Asiatic clam range from 1.2 to 4.7 mg/L. Further research showed that the effectiveness of monochloramine increased greatly as the temperature increased (Cameron et al., 1989b).

### 6.3.2 Points of Application

The formation of monochloramine can be accomplished by first adding ammonia and then chlorine, or vice versa. Ammonia is added first where formation of objectionable taste and odor compounds caused by the reaction of chlorine and organic matter are a concern. However, most drinking water systems add chlorine first in the treatment plant in order to achieve the required concentration and contact time (CT) to meet EPA's SWTR disinfection requirements. Typically, the point of ammonia addition is selected to "quench" the free chlorine residual after a target period of time based on optimizing disinfection versus minimizing DBP formation.

Because the germicidal effectiveness of monochloramine is a factor of 200 less than for free chlorine, extremely long contact times are required for monochloramine to meet EPA disinfection CT requirements. Therefore, if ammonia is added first, a means of ensuring that CT requirements are met must be developed.

#### 6.3.2.1 Impact on Other Treatment Processes

Monochloramine addition impacts other processes at the water treatment facility. These impacts include:

- Ammonia used in the chloramination process can provide nutrient ammonia for nitrifying bacteria growth in the distribution system, which can cause increased nitrate levels in the finished water where systems do not normally test for nitrate.
- Imbalances in chlorine and ammonia concentrations (in greater than an 8 to 1 ratio) can cause breakpoint chlorination reactions to occur when encountered in distribution system
- Monochloramine addition upstream of filters will reduce biological growth on filters. This has a favorable impact on the filters by keeping them clean and reducing the backwash frequency. It also has the undesirable impact of reducing BDOC removal in the filters when the filters are run in a biological mode.

The reader is referred to EPA's *Microbial and Disinfection Byproduct Simultaneous Compliance Guidance Document* (expected to be available in 1999) for additional information regarding the interaction between oxidants and other treatment processes.

# 6.4 Pathogen Inactivation and Disinfection Efficacy

Chloramination of drinking water has been practiced in the United States for nearly 80 years. In addition to achieving disinfection, chloramines have been used by the Denver Water Department for controlling tastes and odors since 1918 (Hazen and Sawyer, 1992). Chloramination has also been found to provide a more stable residual in water distribution system. However, because of its relatively weak disinfecting properties for inactivation of viruses and protozoa pathogens, it is rarely used as a primary disinfectant, and then only with long contact times.

## 6.4.1 Inactivation Mechanisms

The mechanisms by which chloramines inactivate microorganisms have been studied to a lesser degree than chlorine. A study of inactivation of *E. coli* by chloramines concluded that monochloramine readily reacts with four amino acids; cysteine, cystine, methionine and tryptophan ( Jacangelo et. al, 1987) . The mechanism of inactivation for chloramine is therefore thought to involve inhibition of proteins or protein mediated processes such as respiration. Jacangelo further concluded that because of the inconsistency in rate of inactivation monochloramine should have "multiple hits" upon bacterial cells before cell death.

Few studies have been performed to determine the mechanism for viral inactivation. The initial site for destruction of bacteriophage f2 involved the RNA fragment (Olivieri et al., 1980). However, the primary mechanism for poliovirus inactivation by chloramines involved the protein coat (Fujioka et al., 1983). Similar to free chlorine, the mechanism of viral inactivation by chloramine may be dependent on factors such as virus type and disinfectant concentration.

## 6.4.2 Environmental Effects

Several studies have been performed to determine the effect of pH, temperature, and organic and inorganic compounds on the disinfection effectiveness of chloramines. Following is a summary of the affect these parameters have on pathogen inactivation.

### *6.4.2.1 pH*

The effect of pH on disinfection has more to do with the organism than with the disinfectant; however, pH also impacts disinfection efficiency by controlling the chloramine species distribution. Studies have indicated that the disinfection efficacy of monochloramine and dichloramine are not equal. One study showed that the bactericidal properties of dichloramine were superior to that of monochloramine (Esposito, 1974). However, pH may be a compounding factor because changes in pH may alter the physiological response of the organism (Hoff and Geldreich, 1981). Other studies

have shown that monochloramine is superior to dichloramine with regard to virucidal ability (Dorn, 1974; Esposito, 1974; Olivieri et al., 1980). Some evidence suggests that solutions containing approximately equal concentrations of monochloramine and dichloramine may be more microbiocidal than those containing only monochloramine or dichloramine (Weber and Levine, 1944).

#### 6.4.2.2 Temperature

Similar to most of the disinfectants discussed in this report, the bactericidal and viral inactivation efficiency of chloramine increases with increasing temperature. Moreover, the efficiency dramatically decreases under conditions of high pH and low temperature. For example, the inactivation of *E. coli.* is approximately 60 times slower at pH 9.5 and temperatures of 2 and 6°C than at pH 7 and temperatures between 20 and 25°C (Wolfe et al., 1984). Similar results were obtained for poliovirus 1 inactivation (Kelley and Sanderson, 1958).

#### 6.4.2.3 Organic Nitrogen and Other Compounds

In addition to ammonia, free chlorine reacts with organic nitrogen compounds to form a variety of organic chloramines. These organic chloramines are undesirable byproducts because they exhibit little or no microbiocidal activity (Feng, 1966). Studies have indicated that chlorine binds to amine-containing compounds more rapidly than to ammonia (Weil and Morris, 1949; Morris, 1967; Margerum et al., 1978) and that chlorine can be transferred from inorganic chloramines to amine-containing compounds (Margerum et al., 1978; Isaac and Morris, 1980)

Several other reactions may occur which divert chlorine from the formation of chloramines. These reactions can include oxidation of iron, manganese, and other inorganics such as hydrogen sulfide (Hazen and Sawyer, 1992).

### 6.4.3 Disinfection Efficacy

Chloramines are relatively weak disinfectants for virus and protozoa inactivation. As a consequence, it is extremely difficult to meet the SWTR CT criteria for primary disinfection of *Giardia* and viruses using chloramines because very long detention times are needed. However, given the ability of chloramines to provide a stable residual, this form of disinfection appears to be feasible for secondary disinfection protection against microbial growth in distribution systems. The following paragraphs describe the disinfection efficiency of chloramines in terms of bacteria, virus, and protozoa inactivation.

#### 6.4.3.1 Bacteria Inactivation

A series of comprehensive experiments was initiated in the mid 1940s to determine the relative bactericidal effectiveness of free chlorine and inorganic chloramines. Results from these experiments showed conclusively that under relatively demand-free, laboratory-controlled conditions, free chlorine inactivated enteric bacteria much faster that chloramines (Wattie and

Butterfield, 1944). In this experiment, a monochloramine concentration of 0.3 mg/L required 240 minutes of contact time for 3-log inactivation of *E. coli* whereas exposure to 0.14 mg/L free $Cl_2$ required only 5 minutes to achieve the same level of inactivation at the same temperature and pH.

#### 6.4.3.2 Virus Inactivation

According to reports written by Kabler et al. (1960) and the National Research Council (1980), all studies conducted prior to 1944 that compared virucidal potency of free and combined chlorine were inaccurate because the experiments failed to clearly differentiate between free and combined forms of chlorine and because high-chlorine-demand water was used in the experiments.

The majority of the experiments conducted after the mid-1940s has shown that inorganic chloramines require much higher concentrations and considerably longer contact times than free chlorine to achieve comparable levels of virus inactivation. Experiments showed that contact times between 2 and 8 hours were required at concentrations between 0.67 to 1.0 mg chloramines to achieve greater than 2-log inactivation of poliovirus 1 (Mahoney and MK500), poliovirus 2 (MEF), poliovirus 3 (Sackett), coxsackievirus B1, and coxsackievirus B5 (EA 80) (Kelley and Sanderson, 1958 and 1960). In contrast, 0.2 to 0.35 mg/L free $Cl_2$ required 4 to 16 minutes of contact time to achieve comparable levels of inactivation under the same conditions.

#### 6.4.3.3 Protozoa Inactivation

Of the three predominant forms of pathogens (i.e., bacteria, viruses, and protozoan [oo]cysts), studies have shown that protozoan [oo]cysts are usually the most resistant to all forms of disinfection. Studies have indicated that free chlorine is a more effective disinfectant than chloramines for [oo]cyst inactivation (Chang and Fair, 1941; Chang, 1944; Stringer and Kruse, 1970). Chloramine concentrations of 8 mg/L were required for 2-log inactivation of *Entamoeba histolytica* cysts whereas only 3 mg/L of free chlorine was required to obtain the same degree of inactivation (Stringer and Kruse, 1970). Contact times for both disinfectants were 10 minutes.

#### 6.4.3.4 CT Values

CT values for achieving *Giardia* cyst and virus inactivation using chloramines are shown in Table 6-4 and Table 6-5, respectively. Values contained in these tables were obtained from the *Guidance Manual for Compliance with Filtration and Disinfection Requirements for Public Water Systems Using Surface Water Sources* (AWWA, 1991).

CT values shown in Table 6-4 are based on disinfection studies using in vitro excystation of *Giardia lamblia*. CT values shown in Table 6-5 were based on data using preformed chloramines at pH 8. No safety factor was applied to laboratory data used to derive the CT values shown in Table 6-4 and Table 6-5 since chloramination conducted in the field is more effective than using preformed chloramines, since monochloramine tends to degrade with time and some free chlorine is present when forming chloramines which enhances the inactivation process.

**Table 6-4. CT Values for *Giardia* Cyst Inactivation Using Chloramines**

| | Temperature (°C) (mg•min/L) | | | | |
|---|---|---|---|---|---|
| **Inactivation** | **5** | **10** | **15** | **20** | **25** |
| 0.5-log | 365 | 310 | 250 | 185 | 125 |
| 1-log | 735 | 615 | 500 | 370 | 250 |
| 1.5-log | 1,100 | 930 | 750 | 550 | 375 |
| 2-log | 1,470 | 1,230 | 1,000 | 735 | 500 |
| 2.5-log | 1,830 | 1,540 | 1,250 | 915 | 625 |
| 3-log | 2,200 | 1,850 | 1,500 | 1,100 | 750 |

Source: AWWA, 1991.
Values shown in this table are based on a pH range between 6 and 9.

**Table 6-5. CT Values for Virus Inactivation Using Chloramines**

| | Temperature (°C) (mg•min/L) | | | | |
|---|---|---|---|---|---|
| **Inactivation** | **5** | **10** | **15** | **20** | **25** |
| 2-log | 857 | 643 | 428 | 321 | 214 |
| 3-log | 1,423 | 1,067 | 712 | 534 | 356 |
| 4-log | 1,988 | 1,491 | 994 | 746 | 497 |

Source: AWWA, 1991.

# 6.5 DBP Formation

The effectiveness of chloramines to control DBP production depends upon a variety of factors, notably the chlorine-to-ammonia ratio, the point of addition of ammonia relative to that of chlorine, the extent of mixing, and pH.

Monochloramine ($NH_2Cl$) does not produce DBPs to any significant degree, although some dichloroacetic acid can be formed from monochloramine and cyanogen chloride formation is greater than with free chlorine (Jacangelo et al., 1989; Smith et al., 1993; Cowman and Singer, 1994). The inability to mix chlorine and ammonia instantaneously allows the free chlorine to react before the complete formation of chloramines. In addition, monochloramine slowly hydrolyzes to free chlorine in aqueous solution. Therefore, halogenation reactions occur even when monochloramine is formed prior to addition in the treatment process (Rice and Gomez-Taylor, 1986). The closer the chlorine:ammonia ratio is to the breakpoint, the greater the formation of DBPs (Speed et al., 1987). In addition to controlling the formation of DBPs, chloramination results in lower concentrations of a number of the other specific organic halides generated from free chlorine, except for cyanogen chloride (Krasner et al., 1989; Jacangelo et al., 1989). Increased production of cyanogen chloride is observed when monochloramine is used as a secondary disinfectant instead of free chlorine.

The application of chloramines results in the formation of chlorinated organic material, although it occurs to a much lesser degree than from an equivalent dose of free chlorine. Little is known about the nature of these byproducts, except that they are more hydrophilic and larger in molecular size than the organic halides produced from free chlorine (Jensen et al., 1985; Singer 1993).

# 6.6 Status of Analytical Methods

## 6.6.1 Monitoring of Chloramines

There has been little development of analytical procedures for selective determination of monochloramines (Gordon, et al., 1992). Typically, the methods used for chlorine residual measurement are adapted for chloramine measurement. The DBPR promulgated on December 16, 1998 (63 FR 69390) establishes three analytical methods that are acceptable for measuring residual chloramines (combined chlorine). These methods are presented in 40 CFR § 141.131(c) and include:

- Amperometric Titration (Standard Method 4500-Cl D and ASTM Method D 1253-86);
- DPD Ferrous Titrimetric (Standard Method 4500-Cl F); and
- DPD Colorimetric (Standard Method 4500-Cl G).

If approved by the State, systems may also measure chloramines by using DPD colorimetric test kits.

### *6.6.1.1 Amperometric Titrations*

The amperometric titration method is utilized extensively in water treatment laboratories (Gordon, et al., 1992). This method capable of differentiating the three most common forms of chlorine, namely chlorine/hypochlorous acid/hypochlorite ion, monochloramine, and dichloramine, as long as the combined forms are not present in concentrations greater than about 2 mg/L (as $Cl_2$). For higher concentrations, dilution of the samples is required, but differentiation is still possible (Aoki, 1989).

The amperometric titration method is a standard of comparison for the determination of free or combined chlorine. This method is not greatly affected by common oxidizing agents, temperature variations, turbidity, and color (Standard Methods, 1995). Amperometric titration requires a greater degree of skill than colorimetric methods. The differentiation of free chlorine, monochloramine, and dichloramine is possible by control of potassium iodide (KI) concentration and pH during the analysis.

Several methods are commonly used to measure chlorine species utilizing the amperometric titration including forward and back titration (Gordon, et al., 1992). The lower limit of detection of these methods varies depending on the instrumentation used and type of water sample analyzed. The lower limit of detection for commercial amperometric titrating equipment is about 30 µg/L as $Cl_2$ (Sugam, 1983).

Table 6-6 shows the working range, expected accuracy and precision, operator skill level required, interferences, and current status for amperometric method monochloramine analysis comparison.

#### 6.6.1.2 Colorimetric Methods

Over the years, numerous colorimetric methods have been developed to measure free and combined chlorine in aqueous solutions (Gordon, et al., 1992). Not many of these methods would be recommended as the method of choice. Two of the colorimetric methods listed in Standard Methods (1995), are DPD methods. In addition, the colorimetric LCV method modified by Whittle and Lapteff (1974) can be used to measure free and combined chlorine species.

The DPD methods (ferrous titration and colorimetric) are operationally simpler for determining free chlorine than the amperometric titration (Standard Methods, 1995). Procedures are given for estimating separate monochloramine and dichloramine fractions, as well as combined chlorine fractions.

The LCV method modified by Whittle and Lapteff modifies the discontinued Standard Method for LCV. The maximum chlorine concentration that can be determined by this modified procedure, without dilution of the sample, is 10 mg/L as $Cl_2$ (Whittle and Lapteff, 1974).

See Table 6-6 for the working range, expected accuracy and precision, operator skill level required, interferences and current status for colorimetric method monochloramine analysis comparison.

## 6.6.2 Disinfectant Interferences

Interferences to free chlorine may impact the measurement of monochloramine since the methods use the free chlorine level in the determination of monochloramine. Many strong oxidizing agents interfere in the measurement of free chlorine in all monochloramine methods, including bromine, chlorine dioxide, iodine, permanganate, hydrogen peroxide, and ozone. However, the reduced form of these compounds (i.e. bromide ion, chloride ion, iodide ion, manganous ion, and oxygen) do not interfere. Reducing agents such as ferrous compounds, hydrogen sulfide, and oxidizable organic matter generally do not interfere (Standard Methods, 1995).

## Table 6-6. Characteristics and Comparisons of Monochloramine[a] Analytical Methods

| Type of Test | Working Range (mg/L) | Expected Accuracy (± %) | Expected Precision (± %) | Skill Level[b] | Interferences | pH Range | Field Test | Automated Test | Current Status |
|---|---|---|---|---|---|---|---|---|---|
| Amperometric Titration, Forward | 0.1 - 10 | NR | 0 - 10 | 2 | dichloramine, nitrogen trichloride, | pH dependent | Yes | Yes | Recommended |
| Amperometric Titration, Back | 0.1 - 10 | NR | NF | 2 | dichloramine, nitrogen trichloride, | pH dependent | Yes | Yes | Recommended |
| Colorimetric DPD Ferrous Titration | 0.01 - 10 | NR | 2 - 7 | 1 | dichloramine, nitrogen trichloride, oxidizing species | Requires buffer | No | No | Recommended Lab Test |
| Colorimetric DPD | 0.01 - 10 | NR | 5 - 75 | 1 | dichloramine, nitrogen trichloride, oxidizing species | Requires buffer | Yes | No | Recommended Field Test |

Source: Gordon et al., 1992.

Notes: [a] Little actual work has been carried out on selective determination of chloramines. The values reported are from extrapolated studies that had objectives other than the selective determination of chloramines.

[b] Operator skill levels: 1 = minimal, 2 = good technician, 3 = experienced chemist. NR = Not reported in literature cited by referenced source.

#### 6.6.2.1 Amperometric Titrations

The amperometric titration methods are unaffected by dichloramine concentrations in the range of 0 to 9 mg/L as $Cl_2$ in the determination of free chlorine. Nitrogen trichloride, if present, may react partially as free chlorine. The amperometric method will measure organic chloramines as free chlorine, monochloramine, or dichloramine, depending upon the activity of chlorine in the organic sample (Standard Methods, 1995). Dichloramine may also interfere with the measurement of both monochloramine and free chlorine (Marks, et al., 1951). The presence of iodide ion can be a severe problem if the titrator glassware is not washed carefully between determinations (Johnson, 1978).

Manganese dioxide, a common interference in most chlorine analytical procedures, does not interfere in the amperometric measurement of free chlorine (Bongers et al., 1977). However, because of its reaction with iodide ion, added during analysis, manganese dioxide does interfere with the amperometric measurement of combined forms of chlorine such as monochloramine (Johnson, 1978).

#### 6.6.2.2 Colorimetric Methods

Sample color and turbidity may interfere in all colorimetric procedures. In the DPD colorimetric methods, high concentrations of monochloramine interfere with free chlorine determination unless arsenite or thioacetimide are added. In addition, the DPD methods are subject to interference by oxidized forms of manganese unless compensated for by a blank (Standard Methods, 1995). The DPD methods are unaffected by dichloramine concentrations in the range of 0 to 9 mg/L as $Cl_2$ in the determination of free chlorine. Nitrogen trichloride, if present, may react partially as free chlorine. The extent of this interference in the DPD methods does not appear to be significant (Standard Methods, 1995).

In the LCV colorimetric method, Whittle and Lapteff (1974) reported that dichloramine did not interfere with the monochloramine measurement.

### 6.6.3 Chloramine Monitoring for Systems Using Chloramines

Pursuant to 40 CFR §141.132(c)(1), community water systems and non-transient non-community water systems that use chloramines, must measure the residual disinfectant level at the same points in the distribution system, at the same time, and at the same frequency (based on population served) as total coliforms are sampled, as specified in 40 CFR §141.21. These systems may use the results of residual disinfectant concentration sampling conducted under §141.74(b)(6)(i) for unfiltered systems or §141.74(c)(3)(i) for systems which filter, in lieu of taking separate samples. No reduced monitoring allowances exist for these systems.

Compliance with the MRDL of 4.0 mg/L (as chlorine) is based on a running annual arithmetic average, computed quarterly, of monthly averages of all samples collected by the system under §141.132(c)(1). If the average quarterly averages covering any consecutive four-quarter period exceeds the MRDL, the system is in violation of the MRDL and must notify both the public, pursuant

to §141.32, and the State, pursuant to §141.134. Where systems switch between the use of chlorine and chloramines for residual disinfection during the year, compliance is determined by including together all monitoring results of both chlorine and chloramines in calculating compliance.

# 6.7 Operational Considerations

The purpose of this section is to address operational considerations in the use of chloramines in drinking water treatment. Specifically, the following topics are addressed below: the conversion of chloramination from chlorination; the potential operational impacts from chloramination disinfection; and special considerations for chloramination facilities. For a more detailed discussion of chloramine disinfection, refer to "Optimizing Chloramine Treatment" by Kirmeyer et, al. 1993.

## 6.7.1 Conversion to Chloramination from Chlorination

### *6.7.1.1 Planning*

Project planning and preparation are essential to ensure an efficient changeover, maintain a dependable and safe system, and preserve the public confidence in the water purveyor (Skadsen, 1993). Planning and preparation should consider the following aspects:

- Raw water composition and suitability to chloramination;
- Treatment plant and distribution system attributes and monitoring program;
- Employee training;
- Public notification and education; and
- Environmental affects from chloraminated water.

### *6.7.1.2 Preliminary Analysis*

A bench scale study is necessary to identify the water characteristics and to determine if chloramination is suitable. White (1992) describes some of the study objectives and variables to consider. The reaction time to form free chloramine residuals varies for each water source since the reaction rate between chlorine and ammonia nitrogen depends on the water's temperature and pH of the water. The reaction rate is also affected by the chlorine and ammonia nitrogen concentrations. To properly control the reaction time between chlorine and ammonia, the study should use different chlorine:ammonia nitrogen ratios, ammonia feed doses, and contact times.

The amount of ammonia required for chloramine residual disinfection depends on the following factors (Dennis et al., 1991):

- Organic nitrogen in the water;
- Ammonia residual desired in the distribution system; and

- Chloramine residual type and concentration required in the distribution system.

If there is organic nitrogen in the untreated water, the amount of supplemental ammonia required should be carefully determined by subtracting the background ammonia from the desired dose. The dose should also consider the amount of ammonia residual desired in the distribution system. For residual disinfection, approximately 1 to 2 mg/L of ammonia is required (Dennis et al., 1991).

For each specific water, a breakpoint curve should be developed to determine the chloramine residual type required. Monochloramine residuals are preferred for most water distribution systems. Dichloramine and nitrogen trichloride residuals may cause taste and odor problems when concentrations exceed 0.8 mg/L or 0.02 mg/L, respectively. Monochloramines are primarily formed when the theoretical chlorine to ammonia dose ratio is less than 5 to 1 (by weight ratio) and the pH is greater than 7.0 (Dennis et al., 1991). The chloramine residual concentration leaving the treatment plant will vary depending on the size of the distribution system and the chloramine demand exerted by the system. Typical chloramine residuals range from 1 to 4 mg/L (Dennis et al., 1991).

#### *6.7.1.3 The Metropolitan Water District (MWDSC)*

MWDSC of Southern California converted from free chlorine to chloramine disinfection in 1985 to assist its 27 member agencies in complying with the EPA's total trihalomethane regulation. MWDSC serves approximately 15 million people and operates five treatment plants, with a combined capacity of 1,670 MGD. Raw water is taken from two sources: the Colorado River and California state project water.

Prior to the changeover, MWDSC performed extensive investigations into the chemical, microbiological, and engineering aspects of chloramine disinfection. To prepare for the changeover, MWDSC coordinated the efforts among its treatment plants, distribution system reservoirs, laboratory personnel, and management. A formal request for approval to use chloramines as a disinfectant was submitted to the California State Department of Health Services. Next, a series of workshops was held on the engineering, chemical, and microbiological aspects of chloramine disinfection. MWDSC also prepared a manual for the type of chloramination application method and ammonia form selected. Information in the manual included the feed equipment information, project specifications, piping layouts, preliminary analysis, and safety and maintenance issues.

It was essential to notify specific sectors of the public that could be affected by the use of chloramines. MWDSC made its customers aware of the changeover and kept them apprised of the options for preventing adverse reactions through an extensive notification program that involved state and county health departments, appropriate interest groups, and the media.

## 6.7.2 Potential Operational Impacts from Chloramination Disinfection

### *6.7.2.1 Pretreatment*

Ammonia in excess of the required chlorine can promote the growth of nitrifying bacteria in filter beds (i.e., rapid sand filters) (White, 1992). The excess ammonia acts as a nutrient and causes the growth of nitrifying bacteria, which convert the excess ammonia to nitrates and nitrites. Excessive levels of nitrate in drinking water have caused serious illness and sometimes death in infants under six months of age. The symptoms include shortness of breath and blueness of skin [40 CFR §141.32(e)(20)]. Prior to designing a chloramination facility, the amount of ammonia naturally occurring in the raw water should be determined. The required ammonia dosage would then be based on the anticipated naturally occurring ammonia levels.

A chloramine residual concentration should also be maintained in the discharge stream from the filters. An adequate residual concentration would be between 0.5 to 1 mg/L chloramine (White, 1992).

### *6.7.2.2 Nitrification*

Nitrification in chloraminated drinking waters is usually partial. Partial nitrification occurs when the chloraminated water has excess ammonia present in the distribution system (Skadsen, 1993). Partial nitrification can have various adverse effects on water quality, including a loss of total chlorine and ammonia residuals and an increase in heterotrophic plate count (HPC) bacteria concentration. The excess ammonia encourages the growth of nitrifying bacteria that convert ammonia to nitrates. An intermediate step in this conversion results in a small amount of nitrite being formed. Research has shown that a chlorine demand of 5 mg/L is exerted by 1 mg/L of nitrite (Cowman and Singer, 1994). The nitrites rapidly reduce free chlorine, accelerate decomposition of chloramines, and can interfere with the measurement of free chlorine (Skadsen, 1993). Valentine (1998) found that the decay of monochloramine was increased (from a second order rate constant of 0.07 to 0.106) by the presence of 0.5 mg/L of nitrite. If nitrification episodes are allowed to continue, very low (or zero) total chlorine residual concentration levels may occur. Loss of chlorine residual allows an increase in HPC bacteria and potentially increases in total coliforms and may result in a positive sample (Cowman and Singer, 1994). Additional information on nitrification can be found in (Kirmeyer et, al. 1995), “Nitrification Occurrence and Control in Chloraminated Water Systems.”

**Factors.** Several possible factors have been implicated as contributing to nitrification. These factors include low chlorine-to-ammonia ratio, long detention times, and temperatures (Cowman and Singer, 1994). Though some articles noted that low monochloramine dosages may lead to nitrification, other research has reported nitrification occurring at monochloramine concentrations greater than 5.0 mg/L (Cowman and Singer, 1994). Nitrifying bacteria are relatively more resistant to disinfection by monochloramine than free chlorine (Cowman and Singer, 1994). The optimum conditions for nitrification would be a water system with free-ammonia, a pH of 7.5 to 8.5, a water temperature of

25 to 30°C, and a dark environment. Nitrifying bacteria exhibit slow growth and have been found in higher numbers in the sediment of distribution systems than in the biofilm (Cowman and Singer, 1994).

If the water reservoirs in the distribution system are covered, partial nitrification may occur (White, 1992). Nitrification occurred in two of MWDSC's covered reservoirs (Garvey and Orange County reservoirs) after the changeover. Approximately 10 weeks after the changeover from chlorine to chloramine, water quality degradation was occurring in the Garvey Reservoir. MWDSC increased the amounts of free chlorine added to the plant effluent to maintain a 1.5 mg/L monochloramine residual at the reservoir effluent and the chlorine to ammonia ratio was increased from 3:1 to 4:1 to decrease the amount of excess ammonia in the water. These changes were more effective than the flushing programs for the distribution system, which only helped temporarily.

**Control Measures.** Nitrification may pose a potential problem for any utility using monochloramine as a disinfectant (Cowman and Singer, 1994). Thus, nitrification should be carefully assessed and controlled. Nitrification may be controlled by monitoring at strategic locations throughout the distribution system for monochloramine and dichloramine residuals (White, 1992).

The chloramine free residual stability is increased throughout the distribution system when there is increased control of microbial contaminants and decreased bacterial concentrations in the raw water to acceptable levels. Recommended approaches to prevent and control nitrification in the distribution system include (Cowman and Singer, 1994):

- Decreasing the detention time;
- Increasing the pH;
- Decreasing the temperature;
- Decreasing TOC concentrations;
- Increasing chloramines residual;
- Increasing the chlorine-to-ammonia ratio; and
- Decreasing the excess ammonia concentration.

For the distribution system, the system should be evaluated to identify the low-flow or dead-end sections. The detention times in the system should be operationally minimized (Skadsen, 1993). For reservoirs, those with single inlet-outlet configurations should especially be carefully monitored and operated (Skadsen, 1993).

MWDSC stresses the importance of developing nitrification strategy control measures. In particular, a comprehensive monitoring program should be established to alert personnel to implement control measures when required. To control nitrification, MWDSC developed a control strategy where the reservoirs and distribution system were first sampled for nitrite levels (Skadsen, 1993). MWDSC

also decreased the detention times in reservoirs and distribution systems, especially during warmer weather, which helped to keep nitrite levels down.

The chloramination operation was modified to add more chlorine to the reservoir inlet and increase the chlorine-to-ammonia ratio from 3:1 to 5:1 at the plant effluent. The initial 3:1 ratio corresponded with a 1.5 mg/L monochloramine residual and 0.2 mg/L excess ammonia. At these concentrations, the agencies receiving the water had the flexibility of blending the chloraminated water with chlorinated water or adding more chlorine to those sections of the system with long detention times. Increasing the ratio further to 5:1 controlled the nitrification problem by decreasing the amount of free ammonia in the distribution system. Operating at a 5:1 ratio requires more monitoring since an overdose of chlorine can reduce the chloramine residual.

A survey of chloramine users in the United States was conducted in June 1991. This survey showed that the chlorine to ammonia ratio varied from 3:1 to 7:1 (Dennis, et al, 1991). The chloramine residual varied from 0.8 mg/L to 3 mg/L (Dennis, et al, 1991). Table 6-7 shows the results from this survey. The agencies surveyed reported excellent results with secondary chloramine disinfection (Dennis et al, 1991). EPA is in the process of collecting and evaluating chloramine use in the United States as part of the Information Collection Rule (ICR), but until those data are available, the 1991 survey appears to be the most recent national survey of chloramine use.

Each year, MWDSC also added chlorine past the breakpoint to allow a free residual for 30 days. The ideal locations for breakpoint chlorination are at the distribution reservoirs and interconnections. The increased chlorine oxidizes any nitrite and nitrifying bacteria and eliminates the excess ammonia in the distribution system. For larger water systems, MWDSC recommends maintaining chlorination stations throughout the distribution system. Both fixed and mobile chlorinators may be used. Mobile chlorinator units are self-contained and trailer-mounted with evaporators, chlorinators, generator, a booster pump for transport water, and chlorine injectors. They are designed to draw liquid chlorine directly from a 17-ton chlorine trailer and to inject a chlorine solution into the distribution system or reservoir.

Since nitrifying bacteria were found in higher numbers in the sediments of the distribution system than in the biofilm, flushing sediment from the system will help to control nitrification. The addition of a disinfectant (i.e., free or combined chlorine) is required to remove nitrification.

At the Indiana American Water Company, the distribution system is temporarily converted back to free chlorine for scheduled flushing (Lyn et al., 1995). Utilities should evaluate their flushing program to avoid consumer complaints with inappropriate flushing techniques.

#### 6.7.2.3 Taste and Odor

If the chlorine to ammonia-nitrogen ratios are between 3:1 and 5.5:1, disagreeable tastes and odors should be evaluated at the consumer tap (White, 1992).

Fishy tastes and odors (e.g., from source waters and return washwater from the washwater treatment system) can be controlled by a 1-hour contact time with free-chlorine residual of 2 mg/L prior to the addition of ammonia (Dennis et al., 1991). This prechlorination eliminates the fishy taste and odor but may increase the THM concentrations at the plant effluent.

**Table 6-7. Survey of Chloramine Users in the United States**

| Agency | Treatment Capacity | Type of Ammonia | Chlorine: Ammonia Nitrogen ratio | Chloramine Residual (mg/L) | $NH_3$ Injection Point | Nitrification Control Strategies |
|---|---|---|---|---|---|---|
| City of Dallas, TX | 730 mgd; 3 plants | Anhydrous | 5:1 | 2.1 - 2.3 | Presedimenta -tion, Post-filtration | None |
| City of Denver, CO | 600 mgd; 3 plants | Aqueous (30%) | 3:1 | 0.8 - 1.0 | Post-filtration, prior to chlorine addition | None |
| Indianapolis, Water Co., IN | 176 mgd; 4 plants | Anhydrous | 3:1 varies | 1.5 - 2.0 | Post-filtration | Increase ratio in summer |
| Miami-Dade Water Authority, FL | 300 mgd; 3 plants | Anhydrous | 5:1 | 2.7 - 3.0 | 10 ft after chlorine flash mix | 2 weeks free chlorine every November |
| City of Milwaukee, WI | 305 mgd; 2 plants | Anhydrous | 5:1 | 0.8 - 0.9 | Post-filtration | None |
| City of Philadelphia , PA | 530 mgd; 3 plants | Aqueous (30%) | 3:1 | 2.0 | Post-filtration | None |
| City of Portland, OR | 225 mgd | Anhydrous | 7:1 | 1.8 | 70 ft downstream of chlorine in conduit | None |
| Orleans Parish, LA | 300 mgd; 2 plants | Anhydrous | 3:1 | 2.0 - 2.5 | Pre-filtration | None |
| St. Louis Co. Water Authority, MO | 360 mgd; 4 plants | Aqueous (30%) | 4:1 | 2.5 | Concurrent with chlorine at flash mix, post-filtration | None |

Source: Dennis et al.,1991.

## 6.7.3 Special Considerations for Chloramination Facilities

### *6.7.3.1 GAC Filters with Ammonia Addition*

The Ann Arbor Water Treatment Plant in Michigan is a 50 mgd lime softening plant that draws its water from the Huron River (80 to 90 percent) and ground water (10 to 20 percent). When chloramination is applied to the river water, the chlorine is injected into the raw water line

immediately before ammonia is applied. The total chlorine feed averages 3.3 mg/L with an average demand of 2.0 mg/L for the river water.

Evidence of nitrification occurred immediately after a change in treatment from sand to GAC filtration. Prior to the change to GAC, the treatment plant had successfully used monochloramine as both a primary and a final disinfectant. Nitrification was not evident. The GAC received an application of approximately 1.3 mg/L ammonia. This input of ammonia to the filters constituted a nutrient source that allowed nitrifying bacteria to become established and proliferate. The GAC particles have been observed to harbor nitrifying bacteria and nitrification has been observed in GAC beds. Higher nitrifying bacteria levels have been observed in other filter beds as compared with source water. The GAC effluent also showed pronounced seasonal peaks in HPC bacteria from May to July, and percent total coliforms positive from July to August. These seasonal peaks are most probably temperature related. During periods of nitrification, GAC effluent HPC bacteria concentration was steadily decreasing while in the distribution system, HPC bacteria were increasing.

#### 6.7.3.2 Organic Nitrogen

Concentrations of organic nitrogen and ammonia nitrogen as low as 0.3 mg/L may interfere with the chloramination process. The monochloramine residuals will hydrolyze with the organic nitrogen to form organochloramines, which are nongermicidal. This reaction would take about 30 to 40 minutes. After the monochloramine residuals disappear, free ammonia nitrogen reappears. Free ammonia nitrogen is a powerful biological nutrient. Its presence promotes biological instability in that portion of the distribution system. Biological instability usually results in foul tastes and odors plus dirty and/or colored water at the consumers tap (White, 1992).

The free chlorine residual or chloramine residual method may be used to clean an area with biological instability. Of the two methods, the free chlorine residuals method is superior (White, 1992). Free chlorine residuals restore distribution system stability quicker (i.e., a few days for free chlorine versus weeks for chloramines), the clean-up process can be monitored, and the clean-up is complete when the free chlorine residual concentration reaches 85 percent of the free chlorine concentration.

Based on their conversion to chloramination experience, MWDSC recommends that utilities employing chloramines for disinfection monitor for total organic nitrogen levels. When levels are high, the amino acid fraction is also likely to rise. This rise may impair the chloramination disinfection efficiency if high levels of organic nitrogen are not detected.

#### 6.7.3.3 Mixing

Mixing at the point of application greatly affected the bactericidal efficiency of the chloramine process. When the pH of the water is between 7 and 8.5, the reaction time between ammonia and chlorine is practically instantaneous. If chlorine is mixed slowly into the ammoniated water, organic matter, especially organic matter prone to bleaching with chlorine solution, may react with the chlorine and interfere with chloramine formation (White, 1992).

#### 6.7.3.4 Blending Waters

When chlorinated water is blended with chloraminated water, the chloramine residual will decrease after the excess ammonia has been combined and monochloramine is converted to dichloramine and nitrogen trichloride. The entire residual can be depleted. Therefore, it is important to know how much chlorinated water can be blended with a particular chloraminated water stream without significantly affecting the monochloramine residual. Blended residual curves should be developed for each specific blend.

#### 6.7.3.5 Corrosion

Chloramination and corrosion control can limit bacterial biofilm development in the distribution system. If optimum corrosion of iron pipes is not controlled, the chloramination efficiency may be impacted. Corrosion inhibitors with higher phosphate concentrations may reduce corrosion rates (Lyn et al., 1995).

#### 6.7.3.6 Formation of Nitrogen Trichloride

If water in the distribution system tends to form nitrogen trichloride, the finished water should be subjected to post-aeration, which readily removes nitrogen trichloride (White, 1992). Nitrogen trichloride is also readily destroyed by sunlight (White, 1992).

#### 6.7.3.7 Human Health and the Environment

Users of kidney dialysis equipment are the most critical group that can be impacted by chloramine use. Chloramines can cause methemoglobinemia and adversely affect the health of kidney dialysis patients if chloramines are not removed from the dialysate water. Chloramines can also be deadly to fish. The residuals can damage the gill tissues, enter the red blood cells, and cause an acute blood disorder. Chloramine residuals should be removed from the water prior to the water contacting any fish. As such, fish hobbyists should be notified, along with pet stores and aquarium supply establishments.

# 6.8 Summary

## 6.8.1 Advantages and Disadvantages of Chloramine Use

The following list highlights selected advantages and disadvantages of using chloramines as a disinfection method for drinking water (Masschelein, 1992). Because of the wide variation of system size, water quality, and dosages applied, some of these advantages and disadvantages may not apply to a particular system.

**Advantages**

- Chloramines are not as reactive with organics as free chlorine in forming DBPs.

- The monochloramine residual is more stable and longer lasting than free chlorine or chlorine dioxide, thereby providing better protection against bacterial regrowth in systems with large storage tanks and dead end water mains. However excess ammonia in the network may cause biofilming.
- Because chloramines do not tend to react with organic compounds, many systems will experience less incidence of taste and odor complaints when using chloramines.
- Chloramines are inexpensive.
- Chloramines are easy to make.

#### Disadvantages

- The disinfecting properties of chloramines are not as strong as other disinfectants, such as chlorine, ozone, and chlorine dioxide.
- Chloramines cannot oxidize iron, manganese, and sulfides.
- When using chloramine as the secondary disinfectant, it may be necessary to periodically convert to free chlorine for biofilm control in the water distribution system.
- Excess ammonia in the distribution system may lead to nitrification problems, especially in dead ends and other locations with low disinfectant residual.
- Monochloramines are less effective as disinfectants at high pH than at low pH.
- Dichloramines have treatment and operation problems.
- Chloramines must be made on-site.

## 6.8.2 Summary Table

Table 6-8 summarizes the considerations for the use of chloramine.

**Table 6-8. Summary of Chloramine Disinfection**

| Consideration | Description |
|---|---|
| Generation | Chloramines are generated by the sequential addition of chlorine (hypochlorous acid) and ammonia at a $Cl_2$ to $NH_3$ ratio ranging from of 3:1 to 5:1. Either chlorine or ammonia may be added first. Chlorine is normally added first to act as the primary disinfectant and after 10 to 30 minutes, ammonia is added to prevent further formation of DPBs.<br><br>The most common methods of chlorine addition include gas feed using a dilution water eduction system or direct feed of bulk hypochlorite solution (12 percent typical commercial strength).<br><br>The most common ammonia feed facilities include anhydrous ammonia fed either directly or via a dilution water eduction system or direct feed of bulk aqua ammonia solution (20 percent typical commercial strength). |
| Primary uses | Monochloramine is used primarily as a secondary disinfectant to provide a residual in the distribution system. It is used where elevated DBPFP levels in the treated water can cause high levels of DBP formation in the distribution system if free chlorine is used as the secondary disinfectant. Monochloramine has been found to be more effective than free chlorine in controlling biofilms and coliform bacteria in systems with long detention times due the lower decay rate of chloramine. Monochloramine will have much less tendency to react with organics present and hence will form less taste and odor causing compounds. |
| Inactivation efficiency | At pH 7 and below, free chlorine is 200, 200, 50, and 2.5 times more effective in inactivating bacteria, viruses, spores, and cysts respectively than monochloramine. |
| Byproduct formation | Monochloramine substantially reduces the DBP formation but still forms some DBPs. |
| Limitations | Monochloramine is increasingly being used as a secondary disinfectant to provide a residual in distribution systems because of its lower decay rate then free chlorine and lesser tendency to form DBPs.<br><br>Caution should be used in using monochloramine in distribution systems where water sources using free chlorine residual are also used. High $Cl_2$ to N ratios can occur where waters using different residuals combine leading to the possible formation of taste and odor causing dichloramine and nitrogen trichloride. In some cases the residual maybe completely removed by the breakpoint reaction. |
| Point of application | Monochloramine is normally generated at the treatment facility with the addition of ammonia to chlorinated water. Ammonia is normally added prior to the pumping into the distribution system. In some cases, ammonia is added prior to the clearwell to minimize formation of DBPs by free chlorine residual. |
| Special considerations | Nitrification and generation of bacterial growths can occur if the $Cl_2$ to N ratio is too low and conditions exist for the growth of nitrifying bacteria. A minimum residual of 2.0 mg/L of monochloramine has been found effective in controlling nitrification in most systems. |

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# 7. PEROXONE (OZONE/HYDROGEN PEROXIDE)

Advanced oxidation processes generate highly reactive hydroxyl free radicals to oxidize various compounds in the water. As discussed in Chapter 3, hydroxyl radicals are produced during the spontaneous decomposition of ozone. By accelerating the ozone decomposition rate, the hydroxyl radical concentration is elevated, which increases the oxidation rate. This procedure increases the contribution of indirect oxidation over direct ozone oxidation as discussed in Chapter 3.

Several methods have been used to increase ozone decomposition and produce high concentrations of hydroxyl radicals. One of the most common of these processes involves adding hydrogen peroxide to ozonated water, a process commonly referred to as peroxone.

The Metropolitan Water District of Southern California (MWDSC) conducted extensive research into the use of peroxone to control organics and oxidize taste and odor compounds (e.g., geosmin and 2-methylisoborneol [MIB]) while providing sufficient levels of molecular ozone to guarantee CT values and primary disinfection. While this chapter focuses on peroxone as a disinfectant, similar results are expected from other advanced oxidation processes such as ozone plus UV, ozone at high pH, hydrogen peroxide plus UV, and other combinations.

A key issue with the use of peroxone as a disinfection process is that the process does not provide a measurable disinfectant residual. It is therefore not possible to calculate CT similar to the use of other disinfectants. While no credit can be given for hydroxyl free radicals because it cannot be measured directly, some credit may be considered for any detected ozone in peroxone systems. Peroxone does provide pathogen inactivation, as discussed in this chapter, but equivalent CT values or methods of calculating equipment CT credits have not been established at the date of publication of this guidance document.

## 7.1 Peroxone Chemistry

The ozone decomposition cycle is similar to that discussed in Chapter 3. However, the added hydrogen peroxide or ultraviolet radiation accelerates the decomposition of ozone and increases the hydroxyl radical concentration. By adding hydrogen peroxide, the net production of hydroxyl free radicals is 1.0 mole hydroxyl radical per mole ozone.

Similar to the discussion of ozone in Chapter 3, oxidation in the peroxone occurs due to two reactions (Hoigné and Bader, 1978):

- Direct oxidation of compounds by aqueous ozone ($O_{3(aq)}$); and
- Oxidation of compounds by hydroxyl radicals produced by the decomposition of ozone.

The two oxidation reactions compete for substrate (i.e., compounds to oxidize). The ratio of direct oxidation with molecular ozone is relatively slow ($10^{-5}$-$10^{7}M^{-1}sec^{-1}$) compared to hydroxyl radical oxidation ($10^{12}$-$10^{14}M^{-1}sec^{-1}$), but the concentration of ozone is relatively high. On the other hand, the hydroxyl radical reactions are very fast, but the concentration of hydroxyl radicals under normal ozonation conditions is relatively small.

A key difference between the ozone and peroxone processes is that the ozone process relies heavily on the direct oxidation of aqueous ozone while peroxone relies primarily on oxidation with hydroxyl radical. In the peroxone process, the ozone residual is short lived because the added peroxide greatly accelerates the ozone decomposition. However, the increased oxidation achieved by the hydroxyl radical greatly outweighs the reduction in direct ozone oxidation because the hydroxyl radical is much more reactive. The net result is that oxidation is more reactive and much faster in the peroxone process compared to the ozone molecular process. However, because an ozone residual is required for determining disinfection CT credit, peroxone may not be appropriate as a pre-disinfectant.

The peroxone process utilizes oxidation by hydroxyl radicals. The oxidation potential of the hydroxyl radical and ozone are as follows:

$$OH + e^- \rightarrow OH^- \qquad E^0 = +2.8V$$

$$O_3 + 2H^+ + 2e^- \rightarrow O_2 + H_2O \qquad E^0 = +2.07V$$

$$O_3 + H_2O + 2e^- \rightarrow O_2 + 2OH^- \qquad E^0 = +1.24V$$

In addition to having an oxidation potential of hydroxyl radical higher than ozone, the hydroxyl radical is also much more reactive approaching the diffusion control rates for solutes such as aromatic hydrocarbons, unsaturated compounds, aliphatic alcohols, and formic acid (Hoigné and Bader, 1976).

## 7.1.1 Oxidation Reactions

Because the radical oxidation is much more effective than direct oxidation with ozone, it has been used extensively to treat difficult to oxidize organics such as taste and odor compounds and chlorinated organics (e.g., geosmin, MIB, phenolic compounds, trichloroethylene [TCE], and perchloroethylene [PCE]).

Neither ozone nor peroxone significantly destroys TOC. Peroxone will oxidize the saturated organics and produce byproducts similar to those found in ozonation; namely, aldehydes, ketones, peroxides, bromate ion, and biodegradable organics (MWDSC and JMM, 1992). However, with peroxone, the

biodegradibility of the water (not the organic compounds) increases, rendering "a portion of the TOC" amenable to removal in biologically active filters.

Peroxone has found a niche in oxidizing difficult-to-treat organics, such as taste and odor compounds including geosmin and MIB (Pereira et al., 1996; Ferguson et al., 1990). In addition, peroxone and other advanced oxidation processes have been shown to be effective in oxidizing halogenated compounds such as 1,1-dichloropropene, trichloroethylene, 1-chloropentane, and 1,2-dichloroethane (Masten and Hoigné, 1992; Aieta et al., 1988; Glaze and Kang, 1988). Hydroxyl radicals will react with all these compounds plus refractory aliphatics such as alcohols and short-chain acids (Chutny and Kucera, 1974).

The optimum peroxide:ozone dose ratio to maximize hydroxyl radicals' reaction rate can be determined for a specific oxidation application. For instance, the optimum peroxide:ozone dose ratio for TCE and PCE oxidation in a ground water was determined to be 0.5 by weight (Glaze and Kang, 1988). Tests showed that TCE required lower ozone dosages for the same percentage removal compared to PCE.

LADWP conducted pilot studies and operated a 2,000 gpm full scale AOP demonstration plant in 1995. The peroxide:ozone dose ratio used was 0.5 to 0.6. Ground water containing up to 447 mg/L TCE and 163 mg/L PCE was treated to below the respective MCLs. However, bromate ion was formed in excess of the 0.010 mg/L MCL (Karimi et al., 1997).

## 7.1.2 Reactions with Other Water Quality Parameters

As with ozone alone, pH and bicarbonate alkalinity play a major role in peroxone effectiveness (Glaze and Kang, 1988). This role is primarily related to bicarbonate and carbonate competition for hydroxyl radical at high alkalinity and carbonate competition for hydroxyl radical at high pH levels. Also, excessive peroxide can also limit the formation of the hydroxyl radical and reduce the effectiveness of peroxone.

Turbidity alone does not appear to play a role in peroxone effectiveness nor does peroxone appear to remove turbidity. Tobiason et al. (1992) studied the impact of pre-oxidation on filtration and concluded that the pre-oxidation did not improve effluent turbidities, but did appear to increase filter run times because of lower head loss or delayed turbidity breakthrough. Filter effluent turbidities were similar for no-oxidant and pre-oxidant trains.

## 7.1.3 Comparison between Ozone and Peroxone

The key difference between ozone and peroxone is in the primary oxidation mode; that is, direct oxidation or hydroxyl radical oxidation. The reactivities of these compounds create a different effect in the reactions with water constituents and, thus, disinfection effectiveness. Table 7-1 summarizes the

key differences between ozone and peroxone as they relate to their application in drinking water treatment.

**Table 7-1. Comparison between Ozone and Peroxone Oxidation**

| Process | Ozone | Peroxone |
|---|---|---|
| Ozone decomposition rate | "Normal" decomposition producing hydroxyl radical as an intermediate product | Accelerated ozone decomposition increases the hydroxyl radical concentration above that of ozone alone. |
| Ozone residual | 5-10 minutes | Very short lived due to rapid reaction. |
| Oxidation path | Usually direct aqueous molecular ozone oxidation | Primarily hydroxyl radical oxidation. |
| Ability to oxidize iron and manganese | Excellent | Less effective. |
| Ability to oxidize taste and odor compounds | Variable | Good, hydroxyl radical more reactive than ozone. |
| Ability to oxidize chlorinated organics | Poor | Good, hydroxyl radical more reactive than ozone. |
| Disinfection ability | Excellent | Good, but systems can only receive CT credit if they have a measurable ozone residual. |
| Ability to detect residual for disinfection monitoring | Good | Poor. Cannot calculate CT value for disinfection credit. |

## 7.2 Generation

The peroxone process requires an ozone generation system as described in Chapter 3 and a hydrogen peroxide feed system. The process involves two essential steps: ozone dissolution and hydrogen peroxide addition. Hydrogen peroxide can be added after ozone (thus allowing ozone oxidation and disinfection to occur first) or before ozone (i.e., using peroxide as a pre-oxidant, followed by hydroxyl radical reactions) or simultaneously. Addition of hydrogen peroxide following ozone is the best way to operate, however a system cannot obtain a CT credit unless the ozone residual is sufficiently high.

There are two major effects from the coupling of ozone with hydrogen peroxide (Duguet et al., 1985):

- Oxidation efficiency is increased by conversion of ozone molecules to hydroxyl radicals; and
- Ozone transfer from the gas phase to the liquid is improved due to an increase in ozone reaction rates.

The most efficient operation is to add ozone first to obtain CT disinfection credit, followed by peroxide for hydroxyl radical oxidation.

Ozonation can be described as occurring in two stages. In the first stage, ozone rapidly destroys the initial oxidant demand present, thereby enhancing the ozone transfer rate into solution from the gas phase. Addition of hydroxyl free radicals to the first stage should be minimized since the hydrogen peroxide competes with ozone-reactive molecules (i.e., initial demand) for the ozone present. In the second stage, organic matter is oxidized, taking place much slower than in the first stage. Adding hydrogen peroxide during the second stage makes it possible to raise the overall oxidation efficiency, since the reaction of hydrogen peroxide with ozone produces hydroxyl radicals enhancing chemical reaction rates. In practice, the addition of hydrogen peroxide to the second stage of ozonation can be achieved by injecting the hydrogen peroxide into the second chamber of an ozone contactor (Duguet et al., 1985). The most efficient operation is to use ozone first to obtain CT credit and peroxone second for micropollutant destruction.

Energy consumption of the peroxone process includes that for ozone generation and application, plus for metering pumps to feed peroxide. The peroxide addition step does not require any more training from an operator than any other liquid chemical feed system. Systems should be checked daily for proper operation and for leaks. Storage volumes should also be checked daily to ensure sufficient peroxide is on hand, and to monitor usage.

# 7.3 Primary Uses and Points of Application

Peroxone is used for oxidation of taste and odor compounds, and oxidation of synthetic organic compounds. Peroxone is also used for the destruction of herbicides (e.g., atrazine), pesticides, and VOCs. Peroxone is applied at points similar to ozone for oxidation. Addition of ozone first and hydrogen peroxide second is the better way to operate. Alternatively, hydrogen peroxide can be added upstream of ozone.

## 7.3.1 Primary Uses

### *7.3.1.1 Taste and Odor Compound Oxidation*

Peroxone is used to remove taste and odor causing compounds because many of these compounds are very resistant to oxidation, even ozone-oxidation. More recently, significant attention has been given to tastes and odors from specific compounds such as geosmin, 2-methyliosborneol (MIB), and chlorinated compounds. Studies at MWDSC demonstrated the effectiveness of peroxone to remove geosmin and MIB during water treatment (Ferguson et al., 1990; Ferguson et al., 1991; Huck et al., 1995).

### *7.3.1.2 Synthetic Organic Compound Oxidation*

Peroxone and other advanced oxidation processes have been shown to be effective in oxidizing halogenated compounds such as 1,1-dichloropropene (DCPE), TCE, 1-chloropentane (CPA), and 1,2-

dichloroethane (DCA) (Masten and Hoigné, 1992; Aieta et al., 1988; Glaze and Kang, 1988). The hydroxyl radicals formed react with all these compounds plus refractory aliphatics, such as alcohols and short-chain acids (Chutny and Kucera, 1974).

## 7.3.2 Points of Application

The peroxone process is applied at points similar to ozone for oxidation as discussed in Chapter 3. Importantly, peroxone addition should be after settling and prior to biological filtration. It is important to add hydrogen peroxide after the initial ozone demand is consumed to avoid hydroxyl free radical competition with the initial ozone demanding constituents.

### *7.3.2.1 Impact on Other Treatment Processes*

Peroxide addition impacts other processes at the water treatment facility. These impacts include:

- The use of hydroxyl free radicals generates BDOC, which can cause biological growth in distribution systems if not reduced during biologically active filtration. When peroxide addition is placed before filters, it impacts the filters by increasing biological growths and increasing backwash frequency (depending on the level on BDOC produced).
- Hydroxyl free radicals are strong oxidants that interfere with addition of other oxidants, such as chlorine, until the ozone residual is quenched.
- The oxidation of iron and manganese by hydroxyl free radicals generates insoluble oxides that should be removed by sedimentation or filtration. This also may impact the filters by increasing the load on the filters and increasing backwash frequency.

The reader is referred to the *Microbial and Disinfection Byproduct Simultaneous Compliance Guidance Document* (currently in production) for additional information regarding the interaction between oxidants and other treatment processes.

# 7.4 Pathogen Inactivation

Both peroxone and other advanced oxidation processes have been proven to be equal or more effective than ozone for pathogen inactivation. Disinfection credits are typically described in terms of CT requirements. Because peroxone leaves no measurable, sustainable residual, calculation of an equivalent CT for disinfection credit is not possible unless there is measurable ozone residual.

## 7.4.1 Inactivation Mechanism

Experiments have indicated that long contact times and high concentrations of hydrogen peroxide are required for bacteria and virus inactivation (Lund, 1963; Yoshe-Purer and Eylan, 1968; Mentel and Schmidt, 1973). Achieving a 99 percent inactivation of poliovirus required either a hydrogen peroxide dose of 3,000 mg/L for 360 minutes or 15,000 mg/L for 24 minutes. Based on these results, when the combination of ozone and hydrogen peroxide is used, the primary cause for pathogen inactivation is

attributed to ozone, specifically the mechanisms associated with the oxidation of pathogens by direct ozone reaction and hydroxyl radicals.

As described in Chapter 3, the mode of action of ozone on microorganisms is poorly understood. Some studies on bacteria suggest that ozone alters proteins and unsaturated bonds of fatty acids in the cell membrane, leading to cell lysis (Scott and Lesher, 1963; Pryor et al., 1983), while other studies suggest that ozone may affect deoxyribonucleic acid (DNA) in the cell (Hamelin and Chung, 1974; Ohlrogge and Kernan, 1983; Ishizaki et al., 1987). Virus inactivation was reported to be related to the attack of the protein capsid by ozone (Riesser et al., 1977). Little information was found discussing the mode of action of ozone on protozoan oocysts. However, a few researchers have suggested that ozone causes the oocyst density to decrease and alters the oocyst structure (Wickramanayake, 1984; Wallis et al., 1990).

The debate continues regarding the primary mode of action for hydroxyl free radicals. Some researchers believe that ozone disinfection is a result of direct ozone reaction (Hoigné and Bader, 1975; Hoigné and Bader, 1978), while others believe that the hydroxyl radical mechanism for disinfection is the most important mechanism (Dahi, 1976; Bancroft et al., 1984). Studies using ozone-hydrogen peroxide have shown that disinfection of *E. coli* is less effective as the peroxide to ozone ratio increases to above approximately 0.2 mg/mg (Wolfe et al., 1989a; Wolfe et al., 1989b). The decrease in disinfection was believed to be cause by lower ozone residuals associated with higher peroxide to ozone ratios, which indicates that direct ozone reaction is an important mechanism for pathogen inactivation.

## 7.4.2 Environmental Effects

Although the chemistry of the peroxone process is still not completely understood, optimal production of the hydroxyl radical appears to depend on the pH, ozone concentration, ratio of hydrogen peroxide to ozone, contact time, and water composition (Glaze et al., 1987).

### *7.4.2.1 Competing Chemical Reactions*

One disadvantage of the peroxone process is that it involves radical intermediates that are subject to interference from substances that react with hydroxyl radical, decreasing the effectiveness of the process. Alkalinity, bicarbonate, and pH play a major role in the effectiveness of hydroxyl free radicals. This effect is primarily related to bicarbonate competition for hydroxyl radical at high alkalinity and carbonate competition for hydroxyl radical at pH levels higher than 10.3 (see Chapter 3). Lowering the alkalinity prior to the application of the peroxone process may be necessary for water that has a high bicarbonate level. In addition to carbonate and bicarbonate, organic constituents of humic substances have also been found to react with the hydroxide radical (Glaze, 1986).

#### 7.4.2.2 Ratio of Hydrogen Peroxide and Ozone

A study conducted at MWDSC indicated that the performance of peroxone is greatly dependent upon the peroxide:ozone ratio (Wolfe et al., 1989b). Results from previous studies at MWDSC suggested that the optimal ratio for disinfection was less than or equal to 0.3. One of the primary objectives of the 1989 study was to optimize further the process for disinfection by altering peroxide:ozone ratios and contact times. Results from the study indicated that peroxone at a 0.2 ratio of peroxide:ozone was comparable to ozone for disinfection of indicator organisms and *Giardia muris* cysts, and that at higher ratios, disinfection decreased because ozone decreased.

### 7.4.3 Disinfection Efficacy and Pathogen Inactivation

Recent studies have indicated that the disinfection effectiveness of peroxone and ozone are comparable (Wolfe et al., 1989b; Ferguson et al., 1990; Scott et al., 1992). A study conducted by Ferguson et al. (1990) compared the pathogen inactivation capability of peroxone and ozone using MS-2 and f2 coliphages as well as *E. coli.* and heterotrophic plate count (HPC) bacteria. The f2 and MS-2 coliphages were comparable in their resistance to ozone and peroxone. No differences in the amounts of MS-2 or f2 inactivation were apparent when the peroxide:ozone ratio was varied from 0 to 0.3. Results of the *E. coli.* and HPC studies showed that peroxone and ozone also had comparable results in regards to bacteria inactivation.

Table 7-2 lists CT values derived for inactivation of *Giardia muris* cysts by ozone and peroxone from another study conducted by MWDSC. The contact times used for calculating the CT values were based on 10% and 50% breakthrough of tracer compounds in the contactor. Ozone concentrations used for CT were based on the ozone residual and half of the residual and dose. The results of this study suggest that peroxone is slightly more potent than ozone based on the fact that CT values for ozone were greater than for peroxone. However, because ozone decomposes more rapidly in the presence of hydrogen peroxide, higher ozone dosages may be necessary with peroxone to achieve comparable residuals. Moreover, the use of ozone residuals to calculate CT products for peroxone may not take into account other oxidizing species that may have disinfectant capabilities.

**Table 7-2. Calculated CT Values (mg•min/L) for the Inactivation of *Giardia muris***

| Inactivation | Ozone $C_1T_1$[a] | Ozone $C_2T_2$[a] | Peroxone[b] $C_1T_1$[a] | Peroxone $C_2T_2$[a] |
|---|---|---|---|---|
| 90% | 1.6 | 2.8 | 1.2 | 2.6 |
| 99% | 3.4 | 5.4 | 2.6 | 5.2 |

Data obtained from Wolfe et al., 1989b. Results at 14°C.

[a] $C_1$, ozone residual; $C_2$ (ozone dose + ozone residual)/2; $T_1$ and $T_2$ time (in minutes) to reach 10 percent and 50 percent breakthrough, respectively

[b] The $H_2O_2/O_3$ ratio for all results was 0.2.

## 7.5 Disinfection Byproducts

The principal byproducts associated with peroxone are expected to be similar to those for ozonation and are listed in Table 3-9. Additional DBPs could form from reactions with hydroxyl radicals. Peroxone does not form halogenated DBPs when participating in oxidation/reduction reactions with NOM. However, if bromide ion is present in raw water halogenated DBPs may be formed. Similar to ozone, the principal benefit of using peroxone for controlling THM formation appears to be that it eliminates the need for pre-chlorination and allows lower doses of free chlorine or chloramines to be applied later in the process train after precursors have been removed by coagulation, sedimentation, and/or filtration and at lower doses. But peroxone does not reduce the DBPFP.

Based upon studies and findings involving peroxone, there is no beneficial lowering of THMs as long as free chlorine is utilized as a secondary disinfectant, unless the application of peroxone allows chlorine to be applied later in the process train to water containing reduced precursor concentrations. The MWDSC study found that the use of peroxone/chlorine resulted in THM concentrations 10 to 38 percent greater than the use of ozone/chorine. However, the THM concentrations of waters disinfected with peroxone/chloramines and ozone/chloramines were similar (Ferguson et al., 1990).

The use of peroxone as a primary disinfectant and chloramines as a secondary disinfectant can successfully control halogenated DBP formation if bromide ion is not present and adequate CT credit can be established. As with ozone, bromate ion formation is a potential concern with source waters containing bromide ions. The oxidation reaction of bromide ion ($Br^-$) to hypobromite ion ($BrO^-$) and bromite ion ($BrO_2^-$) and subsequently to bromate ion ($BrO_3^-$) occurs due to direct reaction with ozone, intermediate reactions can also occur through hydroxide radical mediated mechanisms if bromide is not present and adequate CT credit can be established (Pereira et al., 1996).

In general, peroxone produces more bromate ion than ozone when similar ozone residuals (CT credits) are achieved (Krasner et al., 1993). On the other hand, when the ozone dosage is kept constant, peroxone has tended to produce comparable amounts of bromate ion as ozone. Although peroxone produces hydroxyl radicals that can increase bromate ion formation, hydrogen peroxide may also reduce the hypobromite ion (produced initially during the ozonation of bromide) back to bromide ion.

A study by MWDSC evaluated the effectiveness of peroxone to control taste and odor, DBPs, and microorganisms (Ferguson et al., 1990). In attempting to optimize the hydrogen peroxide to ozone ratio ($H_2O_2$:$O_3$) and the contact time for the source water, the study found pre-oxidation of source waters followed by secondary disinfection with chloramines was an effective strategy for controlling concentrations of THMs and other DBPs. The study found that the two source waters disinfected with peroxone, with free chlorine as the secondary disinfectant, resulted in THM concentrations ranging from 67 to 160 µg/L. Conversely, using chloramines as a secondary disinfectant resulted in THM concentrations consistently below 3.5 µg/L (Ferguson et al., 1990).

However, if a short free chlorine contact time is applied after biological filtration and before ammonia addition to inactivate heterotrophic plate count bacteria in the effluent of the biologically active filter—THMs and other DBPs will be formed at higher concentrations than from post chloramination alone. Depending on the TOC and bromide ion concentration of the water, as well as the pH of chlorination, temperature, and reaction time, between 2 and 28 µg/L of TTHMs have been formed in experiments conducted at MWDSC (personal communication, 1998).

Bromate formation in conventional ozonation, and advanced oxidation processes combining ozone and hydrogen peroxide, were recently investigated at five water treatment plants in France. The source water bromide ion concentrations ranged from 35 to 130 µg/L. Bromate ion formation during the ozonation step varied from less than 2 to 42 µg/L. In general, advanced oxidation results in greater bromate ion concentrations when compared with conventional ozonation, provided the same ozone residual is maintained for both processes. However, lower concentrations of bromate ion result if the ozone dosage is kept constant between the two processes and the hydrogen peroxide dosage is increased (von Gunten et al., 1996). To reduce bromate ion formation potential, the proposed ozone contactor at Stone Canyon Filtration Plant includes three ozone application points instead of two (Stolarik and Christie, 1997). Thus, when peroxone is used for obtaining CT credit, more bromate ion may form than during ozonation. However, if peroxone is only used for micropollutant destruction, less bromate ion may form than when ozone is used.

## 7.6 Status of Analytical Methods

Hydrogen peroxide in solution reacts with ozone to ultimately form water and oxygen. Consequently, the simultaneous presence of both oxidants is accepted as being only transient (Masschelein et al., 1977). Chapter 3 summarizes ozone analytical methods that can be used for ozone/hydrogen peroxide disinfectant residual monitoring. This section will present the status of analytical methods for hydrogen peroxide only.

### 7.6.1 Monitoring of Hydrogen Peroxide

Standard Methods (1995) does not list procedures for measuring hydrogen peroxide. Gordon et al. (1992) list several methods for hydrogen peroxide analysis including:

- Titration methods;
- Colorimetric methods; and
- Horseradish peroxidase methods.

Table 7-3 shows the working range, expected accuracy and precision, operator skill level required, interferences and current status for hydrogen peroxide analysis.

#### 7.6.1.1 Titration Methods

Two titration methods are available for the analysis of hydrogen peroxide; namely, iodometric and permanganate. Precautions for the iodometric titration include the volatility of iodine, interferences by metals such as iron, copper, nickel, and chromium, and fading titration end points (Gordon et al., 1992). Organic and inorganic substances that react with permanganate will interfere with the permanganate titration.

Titration of hydrogen peroxide with permanganate or iodide ion is not sufficiently sensitive for determining residual concentrations (Masschelein et al., 1977).

#### 7.6.1.2 Colorimetric Methods

The most widespread method for the colorimetric determination of hydrogen peroxide is that based on the oxidation of a Titanium (IV) salt (Masschelein et al., 1977). A yellow complex is formed and measured by absorption at 410 nm. On a qualitative basis, ozone and persulfates do not produce the same colored complex.

The oxidation of the leuco base of phenolphthalein is used as a qualitative test for hydrogen peroxide (Dukes and Hydier, 1964). Sensitivity and precision of the method is sufficient in the range between 5 and 100 μg/L. This low working analytical range makes this method impractical for measuring hydrogen peroxide residual levels. Also, the instability of the color obtained makes the method less suitable for manual use. No interference data are available, but it is expected that other oxidants would interfere (Gordon et al., 1992).

The oxidation of cobalt (II) and bicarbonate in the presence of hydrogen peroxide produces a carbonato-cobaltate (III) complex (Masschelein et al., 1977). This complex has absorption bands at 260, 440, and 635 nm. The 260 nm band has been used for the measurement of hydrogen peroxide. A detection limit of 0.01 mg/L has been reported (Masschelein et al., 1977). Optical interferences are caused by 100 mg/L nitrate and 1 mg/L chlorite ions. Other oxidizing agents do interfere with this method as will any compound with an absorption at 260 nm (Gordon et al., 1992).

## Table 7-3. Characteristics and Comparisons of Hydrogen Peroxide Analytical Methods

| Type of Test | Working Range (mg/L) | Expected Accuracy (± percent) | Expected Precision (± percent) | Skill Level[a] | Interferences | pH Range | Field Test | Automated Test | Current Status |
|---|---|---|---|---|---|---|---|---|---|
| Iodometric Titration | > 10 | 5 | 5 | 2 | Oxidizing Species | Acidic | Yes | No | Currently Used |
| Permanganate Titration | 0.1 - 100 | 5 | 5 | 2 | Oxidizing Species | Acidic | Yes | No | Currently Used |
| Colorimetry, Titanium IV | 0.1 - 5 | NR | NR | 2 | Ozone | Acidic | Yes | No | Not Recommended |
| Colorimetry, Leuco Base of Phenolphthalein | 0.005 - 0.1 | NR | NR | 2 | Ozone | Neutral | Yes | Yes | Not Recommended |
| Colorimetry, Cobalt III/$HCO_3^-$ | 0.01 - 1 | NR | NR | 2 | Ozone | Neutral | Yes | Yes | Not Recommended |
| HRP[b] | $10^{-8}$ - $10^{-5}$ | NR | NR | 2 | Other Peroxides, Ozone | 4.5 - 5.5 | No | Yes | Currently Used |

Notes:

[a] Operator Skill Levels: 1 = minimal, 2 = good technician, 3 = experienced chemist

NR = Not reported in literature cited by referenced source.

Adapted from Gordon et al., 1992

[b] This method can not be used in real peroxone-treated waters where the hydrogen peroxide concentration is significantly higher.

#### 7.6.1.3 Horseradish Peroxidase Methods

Several methods incorporate the chemical reactions between peroxidase and hydrogen peroxide. Horseradish derived peroxidase (HRP) is used most often. The scopoletin procedure is one of the more widely accepted fluorescent methods of low levels of hydrogen peroxide utilizing HRP (Gordon et al., 1992). Again, no information is available on potential interference.

#### 7.6.1.4 Summary

In general, the analytical procedures for hydrogen peroxide in drinking water are impacted by other oxidizing species such as ozone. Three of the methods are currently used, but not recommended for disinfectant residual measurement (Table 7-3). The scopoletin HRP method is the most promising, although additional study of potential interferences is required (Gordon et al., 1992).

## 7.7 Operational Considerations

Peroxide is a strong oxidant and contact with personnel should be avoided. Secondary containment should be provided for storage tanks to contain any spills. Dual containment piping should be considered to minimize the risk of exposure to plant personnel. Storage containers may explode in the presence of extreme heat or fire.

### 7.7.1 Process Considerations

The impacts of the peroxone process are similar to those described for ozone in Chapter 3. Because an additional oxidant is added to the water, the tendency to transform organic carbon compounds to a more biodegradable form may be increased with the addition of peroxide.

### 7.7.2 Space Requirements

The metering pumps used to add peroxide should be housed with adequate space around each pump for maintenance access. These pumps are generally not very large, so space requirements are not significant.

The storage area can range from small where peroxide is obtained in drums, to large tank farms if plant flow is great. As mentioned previously, secondary containment should be provided. Peroxide has a lower freezing point than water. Housing or heat tracing should be provided for storage tanks and exterior piping if extended periods with temperatures below freezing are anticipated.

### 7.7.3 Materials

Peroxide can be stored in polyethylene drums or tanks. The specific gravity is 1.39 for 50 percent peroxide, which should be considered in the design of the tank walls. Acceptable pipe materials for peroxide include 316 stainless steel, polyethylene, CPVC, and Teflon. Gaskets should be Teflon

because natural rubber, Hypalon and EPDM are not resistant to hydrogen peroxide. Metering pumps heads should be constructed of peroxide resistant materials.

Hydrogen peroxide is purchased from chemical suppliers and is commercially available in 35, 50, and 75 percent strengths. Peroxide is supplied in drums or in bulk by tankcar. Price depends on strength and quantity.

Peroxide can be stored onsite, but deteriorates gradually over time even when stored correctly. Peroxide deteriorates rapidly if contaminated and with heat or exposure to certain materials. Peroxide is added to the water with metering pumps to accurately control dose. Pumps should be designed to prevent potential air binding of peroxide off-gas. Multiple pumps should be provided for redundancy. As with any chemical added to water, adequate mixing should be provided.

# 7.8 Summary

## 7.8.1 Advantages and Disadvantages of Peroxone Use (Ozone/Hydrogen Peroxide)

The following list highlights selected advantages and disadvantages of using peroxone as a disinfection method for drinking water. Because of the wide variation of system size, water quality, and dosages applied, some of these advantages and disadvantages may not apply to a particular system.

### Advantages

- Oxidation is more reactive and much faster in the peroxone process compared to the ozone molecular process.
- Peroxone is effective in oxidizing difficult-to-treat organics, such as taste and odor compounds.
- Peroxone processes have been shown to be effective in oxidizing halogenated compounds.
- The tendency to transform organic carbon compounds to a more biodegradable form may be increased with the addition of peroxide.
- Pumps used to house peroxide are not very large; so space requirements are not significant.

### Disadvantages

- Peroxide is a strong oxidant and contact with personnel is extremely dangerous.
- Peroxide can be stored onsite, but deteriorates gradually even when stored correctly.

- Peroxone as a disinfection process does not provide a measurable disinfectant residual. It is therefore not possible to calculate CT similar to the use of other disinfectants.

- Peroxone's ability to oxidize iron and manganese is less effective than that of ozone.

## 7.8.2 Summary Table

Table 7-4 summarizes the considerations relative to peroxone disinfection considerations.

**Table 7-4. Summary of Peroxone Disinfection Consideration**

| Consideration | Description |
|---|---|
| Generation | Because of its instability, ozone must be generated at the point-of-use. Hydrogen peroxide is purchased from chemical suppliers. Hydrogen peroxide can be stored onsite, but is subject to deterioration. |
| Primary uses | Primary use includes chemical oxidation. As an oxidizing agent, peroxone can be used to remove SOC pollutants and increase the biodegradability of organic compounds. Peroxone is an effective disinfectant but its CT credit has not been established. It is highly reactive and does not maintain an appreciable residual for CT credit calculations. Peroxone may be difficult to use for disinfection because it is highly reactive and does not maintain an appreciable ozone residual level. |
| Inactivation efficiency | Peroxone is one of the most potent and effective germicides used in water treatment. It is slightly more effective than ozone against bacteria, viruses, and protozoan cysts. |
| Byproduct formation | Peroxone itself does not form halogenated DBPs; however, if bromide is present in the raw water or if chlorine is added as a secondary disinfectant, halogenated DBPs including bromate may be formed. Other byproducts include organic acids and aldehydes. |
| Limitations | Ideally, ozone should be used as a primary disinfectant prior to peroxone treatment. |
| Point of application | For disinfection, peroxone addition should be after ozonation. Ozone contact should precede addition of hydrogen peroxide. For oxidation, peroxone can be added prior to coagulation/sedimentation or filtration depending on the constituents to be oxidized. |
| Special considerations | Ozone generation is a relatively complex process. Storage of LOX for ozone generation is subject to building and fire codes. Ozone is a highly toxic gas and the ozone production and application facilities must be monitored for ambient ozone. Hydrogen peroxide is a hazardous material requiring secondary containment for storage facilities. |

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# 8. ULTRAVIOLET RADIATION

Unlike most disinfectants, ultraviolet (UV) radiation does not inactivate microorganisms by chemical interaction. UV radiation inactivates organisms by absorption of the light which causes a photochemical reaction that alters molecular components essential to cell function. As UV rays penetrate the cell wall of the microorganism, the energy reacts with nucleic acids and other vital cell components, resulting in injury or death of the exposed cells. There is ample evidence to conclude that if sufficient dosages of UV energy reach the organisms, UV can disinfect water to whatever degree is required. However, there has been some public health concerns with respect to the overall efficiency of UV to disinfect potable water.

Based on the available research literature, it appears that although exceptional for disinfection of small microorganisms such as bacteria and viruses, UV doses required to inactivate larger protozoa such as *Giardia* and *Cryptosporidium* are several times higher than for bacteria and virus inactivation (White, 1992; DeMers and Renner, 1992). As a result, UV is often considered in concert with ozone and/or hydrogen peroxide to enhance the disinfection effectiveness of UV or for ground water where *Giardia* and *Cryptosporidium* are not expected to occur.

## 8.1 UV Chemistry (Photochemical)

### 8.1.1 UV Radiation

UV radiation quickly dissipates into water to be absorbed or reflected off material within the water. As a result, no residual is produced. This process is attractive from a DBP formation standpoint; however, a secondary chemical disinfectant is required to maintain a residual throughout the distribution system, which may be subject to recontamination.

UV radiation energy waves are the range of electromagnetic waves 100 to 400 nm long (between the X-ray and visible light spectrums). The division of UV radiation may be classified as Vacuum UV (100-200 nm), UV-C (200-280 nm), UV-B (280-315 nm) and UV-A (315-400 nm). In terms of germicidal effects, the optimum UV range is between 245 and 285 nm. UV disinfection utilizes either: low-pressure lamps that emit maximum energy output at a wavelength of 253.7 nm; medium-pressure lamps that emit energy at wavelengths from 180 to 1370 nm; or lamps that emit at other wavelengths in a high intensity “pulsed” manner.

## 8.1.2 UV Disinfection Reactions

The degree to which the destruction or inactivation of microorganisms occurs by UV radiation is directly related to the UV dose. The UV dosage is calculated as:

$$D = I \bullet t$$

Where: $D$ = UV Dose, $mW \cdot s/cm^2$

$I$ = Intensity, $mW/cm^2$

$t$ = Exposure time, s

Research indicates that when microorganisms are exposed to UV radiation, a constant fraction of the living population is inactivated during each progressive increment in time. This dose-response relationship for germicidal effect indicates that high intensity UV energy over a short period of time would provide the same kill as a lower intensity UV energy at a proportionally longer period of time.

The UV dose required for effective inactivation is determined by site-specific data relating to the water quality and log removal required. Based on first order kinetics, the survival of microorganisms can be calculated as a function of dose and contact time (White, 1992; USEPA, 1996). For high removals, the remaining concentration of organisms appears to be solely related to the dose and water quality, and not dependent on the initial microorganism density. Tchobanoglous (1997) suggested the following relationship between coliform survival and UV dose:

$$N = f \cdot D^n$$

Where: $N$ = Effluent coliform density, /100mL

$D$ = UV dose, $mW \cdot s/cm^2$

$n$ = Empirical coefficient related to dose

$f$ = Empirical water quality factor

The empirical water quality factor reflects the presence of particles, color, etc. in the water. For water treatment, the water quality factor is expected to be a function of turbidity and transmittance (or absorbance).

## 8.1.3 Process Variables

Since UV radiation is energy in the form of electromagnetic waves, its effectiveness is not limited by chemical water quality parameters. For instance, it appears that pH, temperature, alkalinity, and total

inorganic carbon do not impact the overall effectiveness of UV disinfection (AWWA and ASCE, 1990). However, hardness may cause problems with keeping the lamp sleeves clean and functional. The presence, or addition, of oxidants (e.g., ozone and/or hydrogen peroxide) enhances UV radiation effectiveness. The presence of some dissolved or suspended matter may shield microorganisms from the UV radiation. For instance, iron, sulfites, nitrites and phenols all absorb UV light (DeMers and Renner, 1992). Accordingly, the absorbance coefficient is an indication of this demand and is unique for all waters. As a result, specific "design" parameters vary for individual waters and should be determined empirically for each application.

UV demand of water is measured by a spectrophotometer set at a wavelength of 254 nm using a 1 cm thick layer of water. The resulting measurement represents the absorption of energy per unit depth, or absorbance. Percent transmittance is a parameter commonly used to determine the suitability of UV radiation for disinfection. The percent transmittance is determined from the absorbance (A) by the equation:

$$\text{Percent Transmittance} = 100 \times 10^{-A}$$

Table 8-1 shows corresponding absorbance measurements and percent transmittance measurements for various water qualities.

**Table 8-1. Water Quality and Associated UV Measurements**

| Source Water Quality | Absorbance (absorbance units/cm) | Percent Transmittance |
|---|---|---|
| Excellent | 0.022 | 95% |
| Good | 0.071 | 85 |
| Fair | 0.125 | 75 |

Source: DeMers and Renner, 1992.

Continuous wave UV radiation at doses and wavelengths typically employed in drinking water applications, does not significantly change the chemistry of water nor does it significantly interact with any of the chemicals within the water (USEPA, 1996). Therefore, no natural physiochemical features of the water are changed and no chemical agents are introduced into the water. In addition, UV radiation does not produce a residual. As a result, formation of THM or other DBPs with UV disinfection is minimal (See Section 8.5, *Disinfection Byproducts of UV Radiation*).

## 8.2 Generation

Producing UV radiation requires electricity to power UV lamps. The lamps typically used in UV disinfection consist of a quartz tube filled with an inert gas, such as argon, and small quantities of mercury. Ballasts control the power to the UV lamps.

## 8.2.1 UV Lamps

UV lamps operate in much the same way as fluorescent lamps. UV radiation is emitted from electron flow through ionized mercury vapor to produce UV energy in most units. The difference between the two lamps is that the fluorescent lamp bulb is coated with phosphorous, which converts the UV radiation to visible light. The UV lamp is not coated, so it transmits the UV radiation generated by the arc (White, 1992).

Both low-pressure and medium-pressure lamps are available for disinfection applications. Low-pressure lamps emit their maximum energy output at a wavelength of 253.7 nm, while medium pressure lamps emit energy with wavelengths ranging from 180 to 1370 nm. The intensity of medium-pressure lamps is much greater than low-pressure lamps. Thus, fewer medium pressure lamps are required for an equivalent dosage. For small systems, the medium pressure system may consist of a single lamp. Although both types of lamps work equally well for inactivation of organisms, low-pressure UV lamps are recommended for small systems because of the reliability associated with multiple low-pressure lamps (DeMers and Renner, 1992) as opposed to a single medium pressure lamp, and for adequate operation during cleaning cycles.

Recommended specifications for low-pressure lamps (DeMers and Renner, 1992) include:

- L-type ozone-free quartz;
- Instant start (minimal delay on startup);
- Designed to withstand vibration and shock; and
- Standard nonproprietary lamp design.

Typically, low-pressure lamps are enclosed in a quartz sleeve to separate the water from the lamp surface. This arrangement is required to maintain the lamp surface operating temperature near its optimum of 40$^{o}$C. Although Teflon sleeves are an alternative to quartz sleeves, quartz sleeves absorb only 5 percent of the UV radiation, while Teflon sleeves absorb 35 percent (Combs and McGuire, 1989). Therefore, Teflon sleeves are not recommended.

## 8.2.2 Ballasts

Ballasts are transformers that control the power to the UV lamps. Ballasts should operate at temperatures below 60$^{o}$C to prevent premature failure. Typically, the ballasts generate enough heat to warrant cooling fans or air conditioning (White, 1992).

Two types of transformers are commonly used with UV lamps; namely, electronic and electromagnetic. Electronic ballasts operate at a much higher frequency than electromagnetic ballasts, resulting in lower lamp operating temperatures, less energy use, less heat production, and longer ballast life (DeMers and Renner, 1992).

Typical ballast selection criteria include (DeMers and Renner, 1992):

- Underwriter's Laboratories (UL) approval;
- Compatibility with UV lamps; and
- Waterproof enclosure in remote location.

### 8.2.3 UV Reactor Design

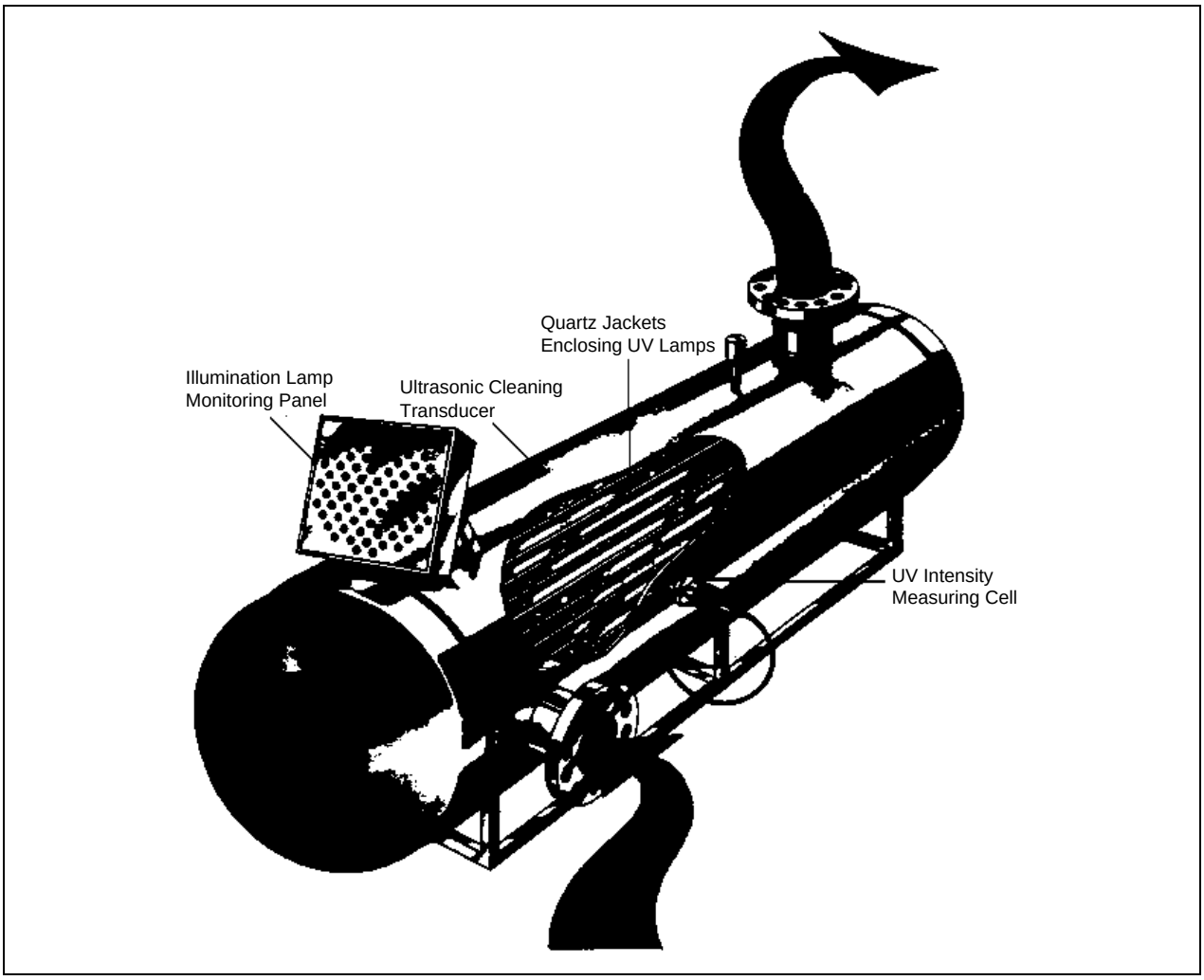
Most conventional UV reactors are available in two types; namely, closed vessel and open channel. For drinking water applications, the closed vessel is generally the preferred UV reactor for the following reasons (USEPA, 1996):

- Smaller footprint;
- Minimized pollution from airborne material;
- Minimal personnel exposure to UV; and
- Modular design for installation simplicity.

Figure 8-1 shows a conventional closed vessel UV reactor. This reactor is capable of providing UV dosages adequate to inactivate bacteria and viruses for flows up to 600 gallons per minute. However, it is incapable of the higher dosages required for protozoan cysts. To increase the dosage, either the number of UV lamps and/or the exposure time should be increased.

Additional design features for conventional UV disinfection systems include:

- UV sensors to detect any drop in UV lamp output intensity;
- Alarms and shut-down systems;
- Automatic or manual cleaning cycles; and
- Telemetry systems for remote installations.




Source: USEPA, 1996.

**Figure 8-1. Closed Vessel Ultraviolet Reactor**

In addition to conventional UV systems, two other UV processes are currently being evaluated for drinking water disinfection:

- Micro-screening/UV; and
- Pulsed UV

Both of these systems profess to provide sufficient UV dose to inactivate *Giardia* cysts and *Cryptosporidium* oocysts. See Section 8.2.3.2, *Emerging UV Reactor Designs*, for further discussion on these emerging technologies.

### *8.2.3.1 Hydraulic Design Considerations*

The major elements that should be considered in the hydraulic design of a UV closed vessel reactor are: dispersion, turbulence, effective volume, residence time distribution, and flow rate (USEPA, 1996).

### Dispersion

Dispersion is the characteristic of water elements to scatter spatially. The ideal UV reactor is plug flow, where water particles are assumed to discharge from the reactor in the same sequence they entered and each element of water passing through the reactor resides in the reactor for the same period of time. An ideal plug flow reactor has no dispersion and is approximated by a long tank with high length-to-width ratio in which dispersion is minimal (USEPA, 1996).

### Turbulence

In addition to plug flow characteristics, the ideal UV reactor has a flow that is turbulent radially from the direction of flow, to eliminate dead zones. This radially turbulent flow pattern promotes uniform application of UV radiation. A negative of having a radially turbulent flow pattern is that some axial dispersion results, thus disrupting the plug flow characteristics. Techniques such as misaligning the inlet and outlet, and using perforated stilling plates, have been used to accommodate the contradicting characteristics of plug flow and turbulence (USEPA, 1996).

## *8.2.3.2 Emerging UV Reactor Designs*

Two emerging technologies in UV reactor design are discussed below. All testing of these two systems to date has been under controlled laboratory or field conditions. Both of these technologies require demonstration of efficacy and applicability in real-world treatment operations.

### Micro-Screening/UV

This unit consists of two treatment chambers, each containing a 2 μm nominal porosity metal screen. Each side of the screen has three 85 watt low pressure mercury lamps, with a total of six lamps per filter. The theoretical minimum dose is 14.6 mW·sec/cm$^2$ at the wavelength of 254 nm. The system is designed to capture *Cryptosporidium* oocysts from the water onto the first screen. The first cycle is set to establish the UV dose. The flow within the first screen chamber is then reversed to backflush to oocysts onto the second screen where the oocysts are trapped until the preset UV dose is reached. Using valves, the pattern of flow is designed to ensure that oocysts are temporarily captured on both filters so that they are exposed to a total preset UV dose, which is totally independent of treated water flowrate (Clancy et al., 1997). Johnson (1997) stated that such a system is capable of achieving total UV doses of 8,000 mW·sec/cm$^2$, sufficient to inactivate *Giardia* cysts and *Cryptosporidium* oocysts. One drawback of this type of reactor is the headloss created (up to 65 feet) by the 2 μm openings in the screen.

### Pulsed UV

In the pulsed UV reactor, capacitors build up and deliver electricity in pulses to xenon flash tubes in the center of a 2 inch diameter flash chamber through which water flows. The unit is designed to provide microsecond pulses at 1 to 30 hertz (or per second). With each pulse, the flash tubes give off high intensity, broad band radiation, including germicidal UV radiation, which irradiates the flowing

water with irradiences of 75 mW·sec/cm$^2$ at 2 cm from the flash tube surface (Clancy et al., 1997). The UV dose can be adjusted by increasing or decreasing the frequency of the pulsing.

# 8.3 Primary Uses and Points of Application

The primary use of UV radiation is to inactivate pathogens to regulated levels. UV radiation is a physical disinfectant that leaves no residual. Therefore, it should be used only as a primary disinfectant followed by a chemical secondary disinfectant to protect the distribution system against coliform proliferation and biofilm formation. See Section 8.4, *Pathogen Inactivation and Disinfection Efficiency*," for a detailed discussion of disinfection efficiency and limitations.

The most common point of application for UV radiation is the last step in the treatment process train just prior to the distribution system and after filtration. The use of UV disinfection has no impact on other processes at the water treatment facility.

# 8.4 Pathogen Inactivation and Disinfection Efficiency

As opposed to most alternative disinfectants, UV is a physical process that requires a contact time on the order of seconds to accomplish pathogen inactivation (Sobotka, 1993). As with any disinfectant, UV has its limitations. For example, because it is a physical rather than a chemical disinfectant, it does not provide a residual to control pathogen proliferation and biofilm formation in the distribution system. When using UV for primary disinfection, some form of secondary chemical disinfectant is required to maintain water quality within the distribution system.

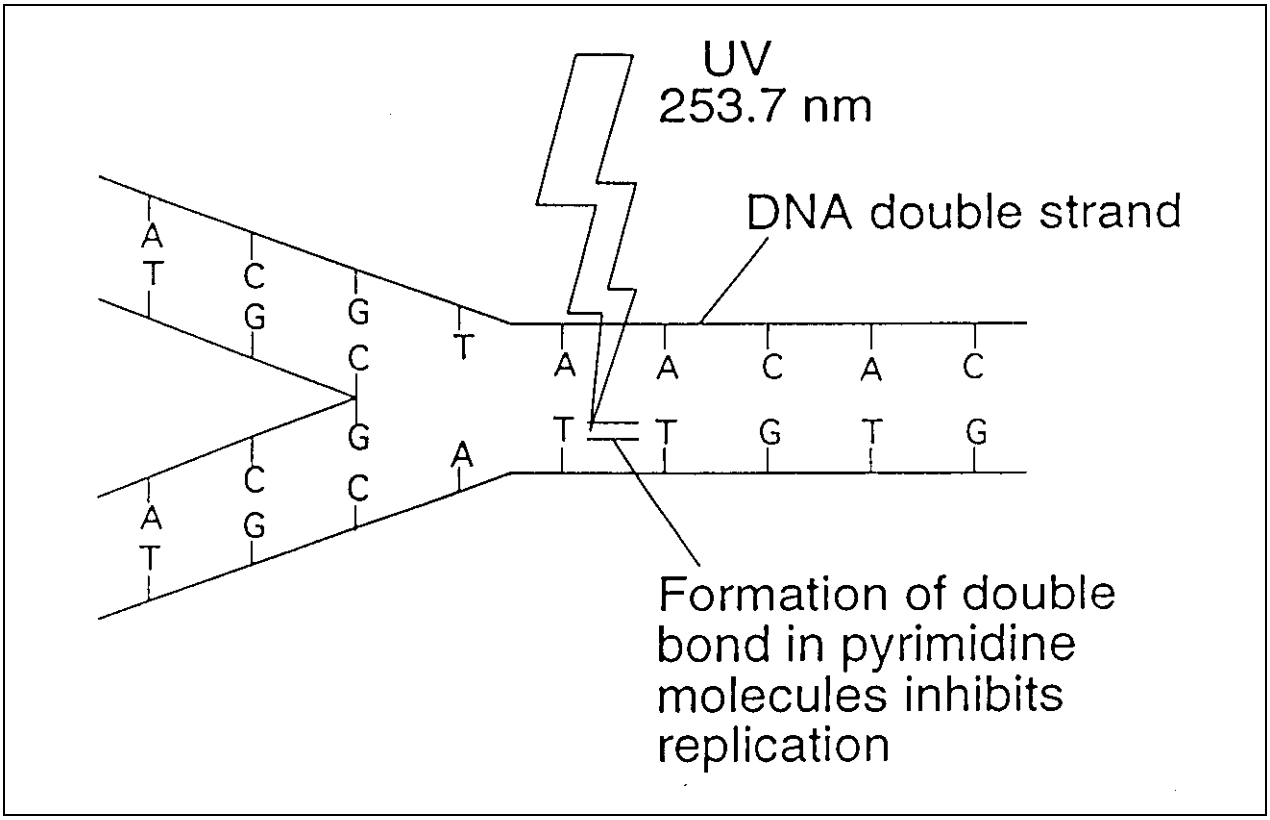
## 8.4.1 Inactivation Mechanism

UV radiation is efficient at inactivating vegetative and sporous forms of bacteria, viruses, and other pathogenic microorganisms. Electromagnetic radiation in the wavelengths ranging from 240 to 280 nanometers (nm) effectively inactivates microorganisms by irreparably damaging their nucleic acid. The most potent wavelength for damaging deoxyribonucleic acid (DNA) is approximately 254 nm (Wolfe, 1990). Other UV wavelengths, such as 200 nm, have been shown to exhibit peak absorbance in aqueous solutions of DNA (von Sonntag and Schuchmann, 1992); however, there is no practical application for UV inactivation of microorganisms in the wavelength range from 190 to 210 nm (USEPA, 1996).

The germicidal effects of UV light involve photochemical damage to RNA and DNA within the microorganisms. Microorganism nucleic acids are the most important absorbers of light energy in the wavelength of 240 to 280 nm (Jagger, 1967). DNA and RNA carry genetic information necessary for reproduction; therefore, damage to either of these substances can effectively sterilize the organism. Damage often results from the dimerization of pyrimidine molecules. Cystosine (found in both DNA and RNA), thymine (found only in DNA), and uracil (found only in RNA) are the three primary types of pyrimidine molecules. Replication of the nucleic acid becomes very difficult once the pyrimidine molecules are bonded together due to the distortion of the DNA helical

structure by UV radiation (Snider et al., 1991). Moreover, if replication does occur, mutant cells that are unable to replicate will be produced (USEPA, 1996). Figure 8-2 is a schematic of the germicidal inactivation observed with UV radiation.

Two phenomena of key importance when using UV disinfection in water treatment are the dark repair mechanisms and the capability of certain organisms to photoreactivate following exposure to certain light wavelengths. Under certain conditions, some organisms are capable of repairing damaged DNA and reverting back to an active state in which reproduction is again possible. Typically, photoreactivation occurs as a consequence of the catalyzing effects of sunlight at visible wavelengths outside of the effective disinfecting range. The extent of reactivation varies among organisms. Coliform indicator organisms and some bacterial pathogens such as *Shigella* have exhibited the photoreactivation mechanism; however, viruses (except when they have infected a host cell that is itself photoreactive) and other types of bacteria cannot photoreactivate (USEPA, 1980; USEPA, 1986; Hazen and Sawyer, 1992). Because DNA damage tends to become irreversible over time, there is a critical period during which photoreactivation can occur. To minimize the effect of photoreactivation, UV contactors should be designed to either shield the process stream or limit the exposure of the disinfected water to sunlight immediately following disinfection.




Source: Tchobanoglous, 1997.

**Figure 8-2. Germicidal Inactivation by UV Radiation**

## 8.4.2 Environmental Effects

To achieve inactivation, UV should be absorbed into the microorganism. Therefore, anything that prevents UV from reacting with the microorganism will decrease the disinfection efficiency. Scheible and Bassell (1981) and Yip and Konasewich (1972) reported that pH had no effect on UV disinfection. Several factors that are known to affect disinfection efficiency of UV are:

- Chemical and biological films that develop on the surface of UV lamps;
- Dissolved organics and inorganics;
- Clumping or aggregation of microorganisms;
- Turbidity;
- Color; and
- Short-circuiting in water flowing through the UV contactor.

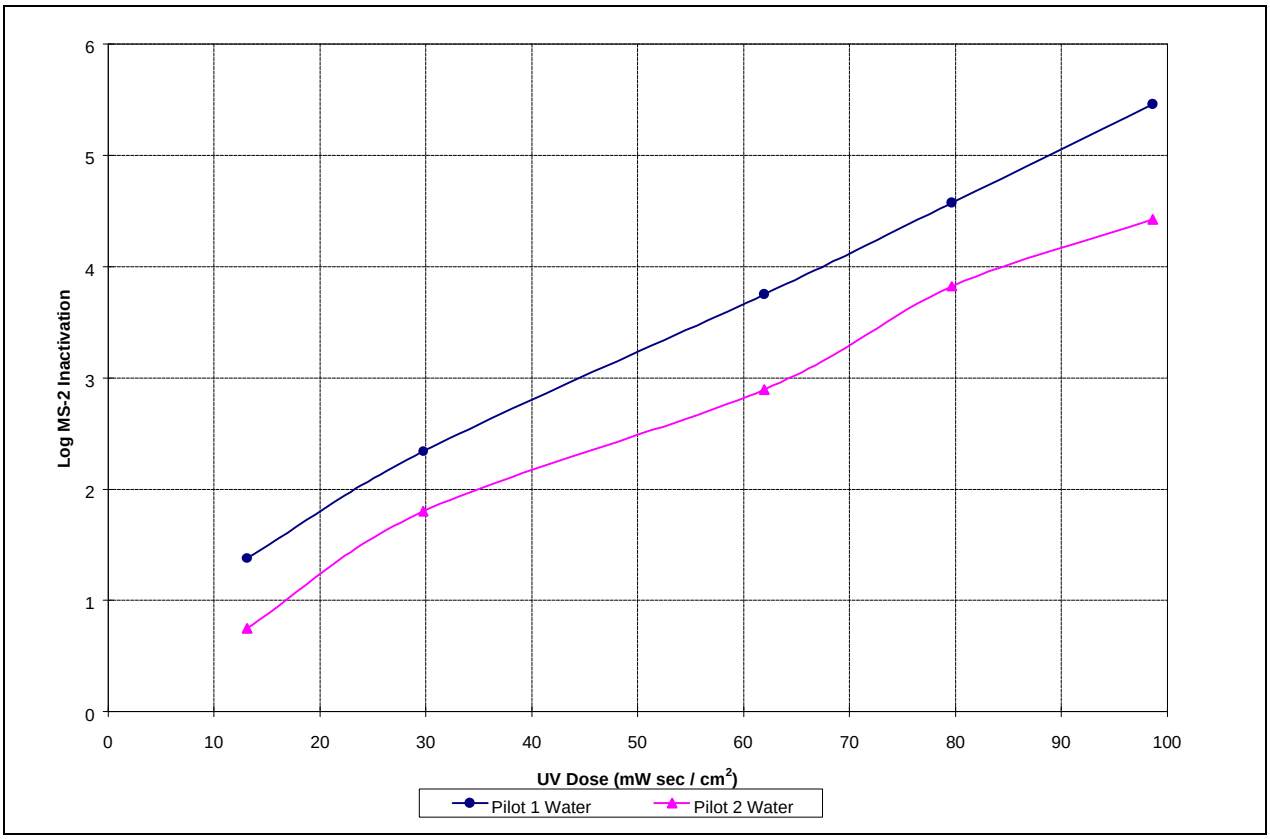
### *8.4.2.1 Chemical Films and Dissolved Organics and Inorganics*

Accumulation of solids onto the surface of the UV sleeves can reduce the applied UV intensity and, consequently, disinfection efficiency. In addition to biofilms caused by organic material, buildup of calcium, magnesium, and iron scales have been reported (DeMers and Renner, 1992). Waters containing high concentrations of iron, hardness, hydrogen sulfide, and organics are more susceptible to scaling or plating (i.e., the formation of a thin coat on unit surfaces), which gradually decreases the applied UV intensity. Scaling is likely to occur if dissolved organics are present and inorganic concentrations exceed the following limits (DeMers and Renner, 1992):

- Iron greater than 0.1 mg/L;
- Hardness greater than 140 mg/L; and
- Hydrogen sulfide greater than 0.2 mg/L.

Figure 8-3 shows the UV dose required for inactivation of MS-2 coliphage at two pilot plants. Snicer et al. (1996) concluded that one possible explanation for higher UV dose for the same degree of inactivation required at pilot plant 2 could be the amount of scaling caused by higher iron concentrations experienced at this plant. Iron concentrations at pilot plant 2 were in the range of 0.45 to 0.65 mg/L, which exceed the limit shown above.

A variety of chemical substances can decrease UV transmission (Yip and Konasewich, 1972), including humic acids, phenolic compounds, and lignin sulfonates (Snider et al., 1991), as well as chromium, cobalt, copper, and nickel. It has been reported that coloring agents, such as Orzan S, tea, and extract of leaves reduce intensity within a UV contactor (Huff, 1965). In addition, iron, sulfites, nitrites, and phenols can absorb UV (DeMers and Renner, 1992).




Source: Snicer et al., 1996.

**Figure 8-3. UV Dose Required for Inactivation of MS-2 Coliphage**

### *8.4.2.2 Microorganism Clumping and Turbidity*

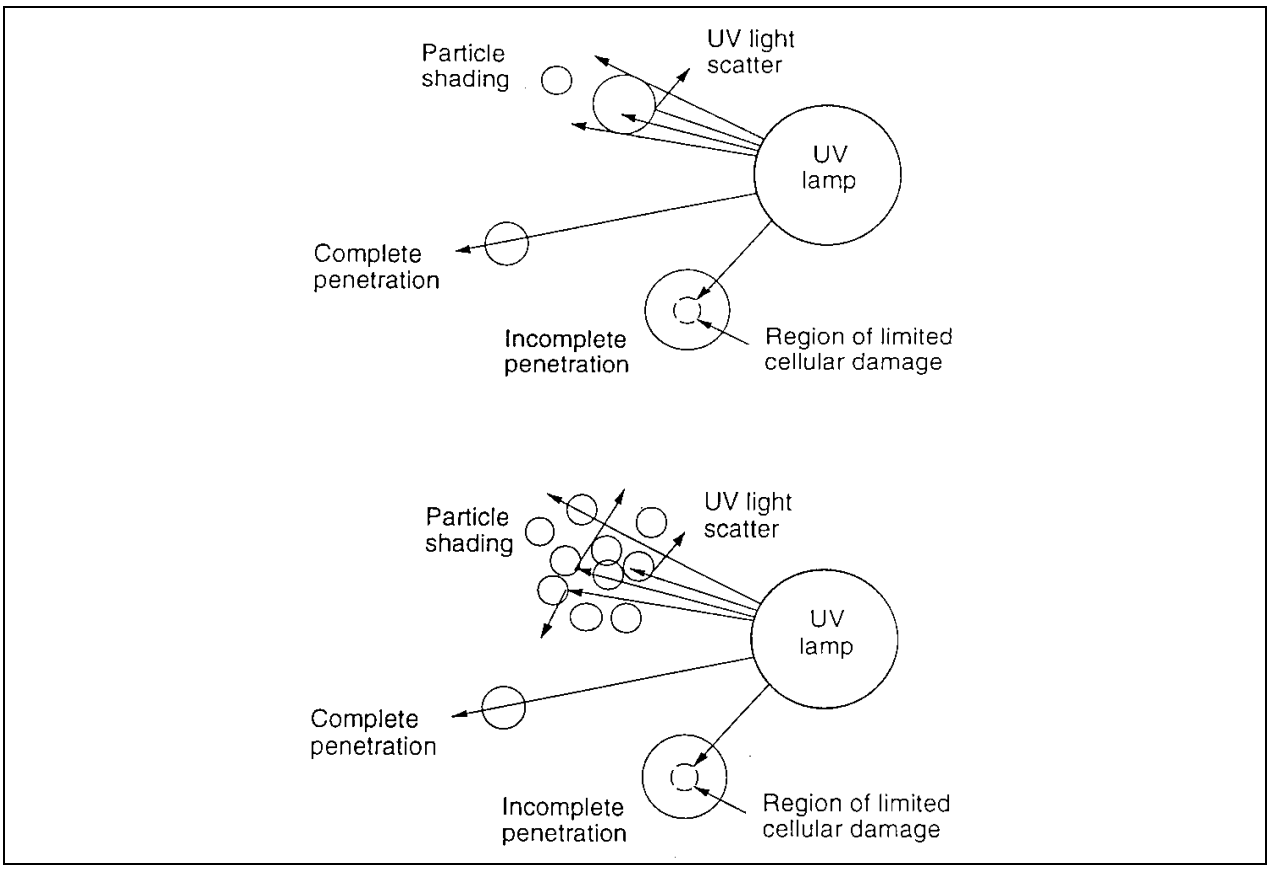
Particles can affect the disinfection efficiency of UV by harboring bacteria and other pathogens, partially protecting them from UV radiation, and scattering UV light (see Figure 8-4). Typically, the low turbidity of ground water results in minimal impact on disinfection efficiency. However, the higher turbidities of surface water can impact disinfection efficiency.

Similar to particles that cause turbidity, microorganism aggregation can impact disinfection efficiency by harboring pathogens within the aggregates and shade pathogens that would otherwise be inactivated.

### *8.4.2.3 Reactor Geometry and Short Circuiting*

Poor geometry within the UV contactor (which creates spacing between lamps) can leave dead areas where inadequate disinfection occurs (Hazen and Sawyer, 1992). A key consideration to improving disinfection is to minimize the amount of dead spaces where limited UV exposure can occur. Plug flow conditions should be maintained in the contactor; however, some turbulence should be created between the lamps to provide radial mixing of flow. In this manner, flow can be uniformly distributed through the varying regions of UV intensity, allowing exposure to the full range of available UV radiation (Hazen and Sawyer, 1992).

As mentioned earlier, UV systems typically provide contact times on the order of seconds. Therefore, it is extremely important that the system configuration limit the extent of short circuiting.




Source: Tchobanoglous, 1997.

**Figure 8-4. Particle Interactions that Impact UV Effectiveness**

## 8.4.3 Disinfection Efficacy

UV disinfection has been determined to be adequate for inactivating bacteria and viruses. Most bacteria and viruses require relatively low UV dosages for inactivation, typically in the range of 2 to 6 $mW \cdot s/cm^2$ for 1-log inactivation. Protozoan [oo]cysts, in particular *Giardia* and *Cryptosporidium*, are considerably more resistant to UV inactivation than other microorganisms. Results of several studies investigating the ability of UV to inactivate bacteria, viruses, and protozoa, are described in the following sections.

### *8.4.3.1 Bacteria and Virus Inactivation*

UV doses required for bacteria and virus inactivation are relatively low. One study determined that UV was comparable to chlorination for inactivation of heterotrophic plate count bacteria following treatment using granular activated carbon (Kruithof et al., 1989).

A study of the ability of UV and free chlorine to disinfect a virus-containing ground water showed that UV is a more potent virucide than free chlorine, even after the chlorine residual was increased to 1.25 mg/L at a contact time of 18 minutes (Slade et al., 1986). The UV dose used in this study was 25 $mW \cdot s/cm^2$.

Table 8-2 shows results from a more recent pilot plant study (Snicer et al., 1996). As indicated by the different UV doses to obtain the same level of inactivation, water characteristics dramatically impact disinfection efficiency. They believed that the higher concentration of iron in pilot plant 2 water (Figure 8-3) interfered with UV or influenced the aggregation of MS-2 viral particles.

Snicer et al. (1996) also compared the susceptibility of MS-2 coliphage to hepatitis A virus, poliovirus, and rotavirus for 10 ground water sources. Results indicated that MS-2 was found to be approximately 2 to 3 times more resistant to UV disinfection than the three human pathogenic viruses.

**Table 8-2. Doses Required for MS-2 Inactivation**

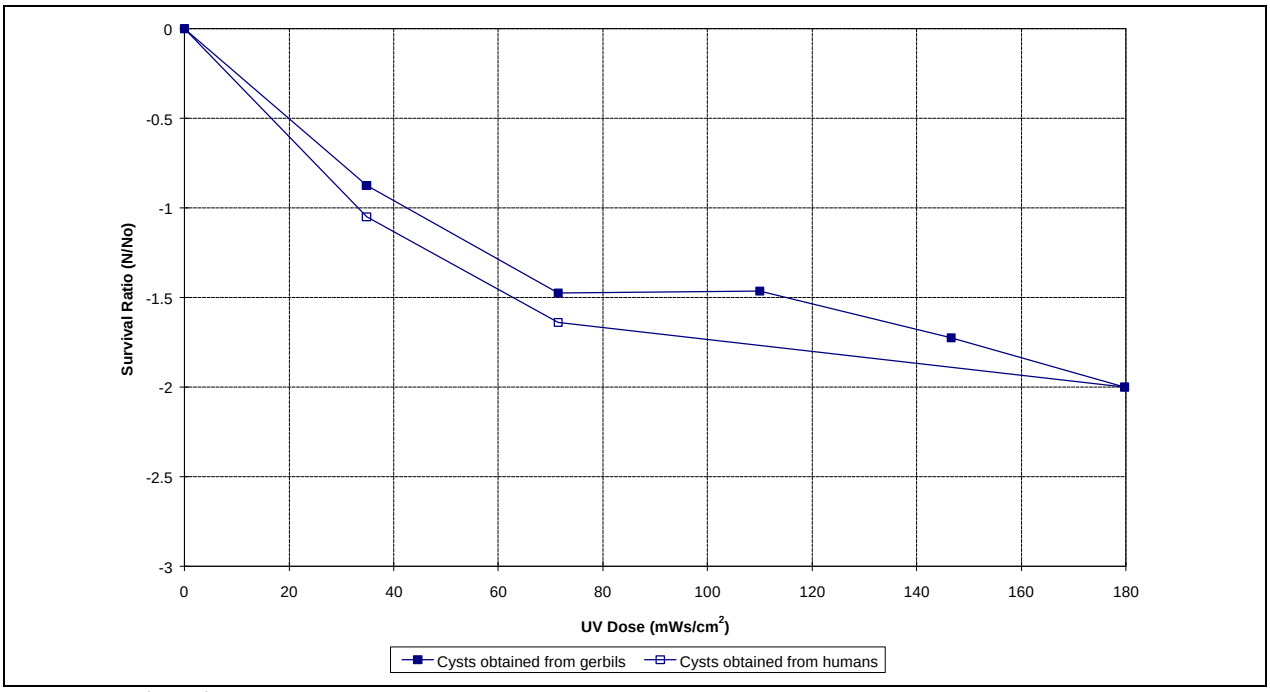
| Log MS-2 Inactivation | Pilot Plant 1 ($mW \cdot s/cm^2$) | Pilot Plant 2 ($mW \cdot s/cm^2$) |
|---|---|---|
| 1 | 3.9 | 15.3 |
| 2 | 25.3 | 39.3 |
| 3 | 46.7 | 63.3 |
| 4 | 68 | 87.4 |
| 5 | 89.5 | 111.4 |
| 6 | 111.0 | 135.5 |

Source: Snicer et al., 1996.

### *8.4.3.2 Protozoa Inactivation*

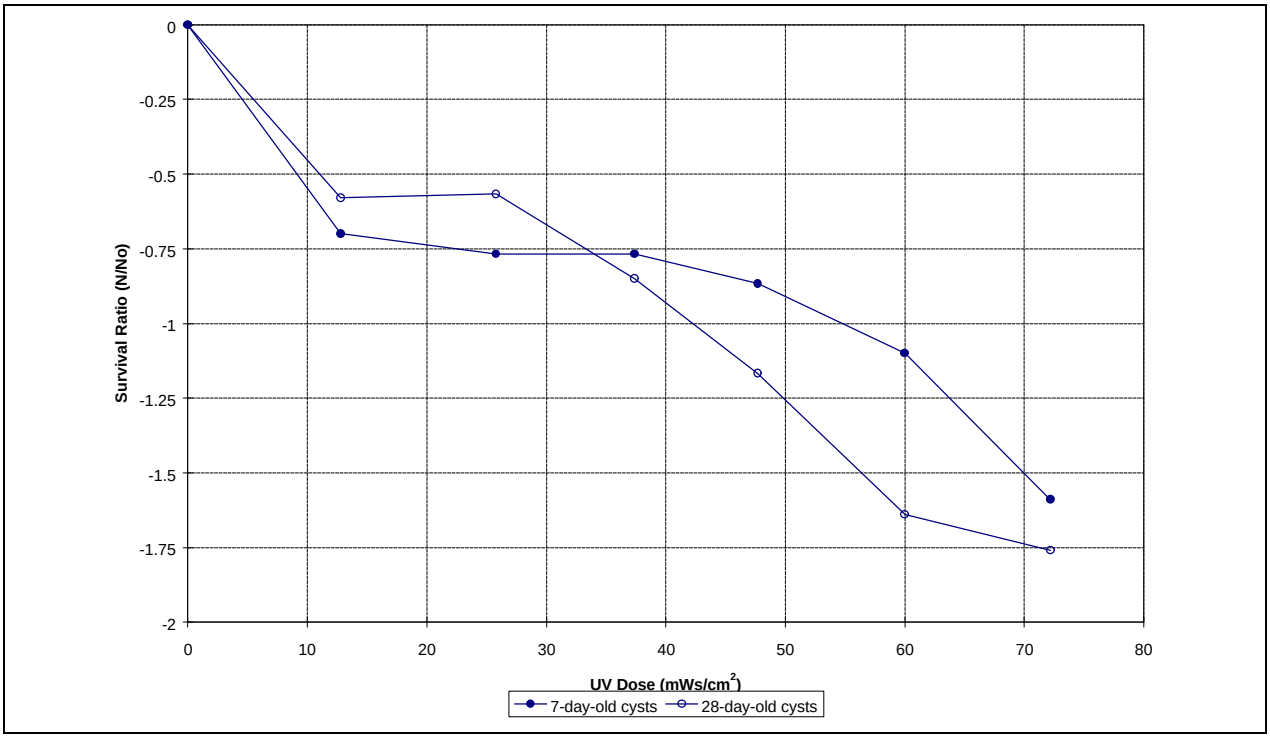
Even though protozoa were once considered resistant to UV radiation, recent studies have shown that ultraviolet light is capable of inactivating protozoan parasites. However, results indicate that these organisms require a much higher dose than that needed to inactivate other pathogens. Less than 80 percent of *Giardia lamblia* cysts were inactivated at UV dosages of 63 $mW \cdot s/cm^2$ (Rice and Hoff, 1981). A 1-log inactivation of *Giardia muris* cysts was obtained when the UV dose was increased to 82 $mW \cdot s/cm^2$ (Carlson et al., 1982).

To achieve 2-log inactivation of *Giardia muris* cysts, a minimum ultraviolet light dose of above 121 $mW \cdot s/cm^2$ is needed. Karanis et al. (1992) examined the disinfection capabilities of ultraviolet light against *Giardia lamblia* cysts extracted from both animals and humans (Karanis et al., 1992). Both groups suffered a 2-log reduction at UV doses of 180 $mW \cdot s/cm^2$. Two important factors to consider when determining dose requirements for *Giardia* inactivation are the parasite source and the growth stage of the microorganism (Karanis et al., 1992). Figure 8-5 shows that the source of the parasites is important in determining dose requirements. Figure 8-6 is from a study conducted in 1992 on *Acanthamoeba rhysodes* inactivation (Karanis et al., 1992). These data show that the age of the protozoa can dramatically affect the dose required to achieve a desired level of inactivation.




Source: Karanis et al., 1992.

**Figure 8-5. UV Doses Required to Achieve Inactivation of *Giardia lamblia* Cysts Obtained from Two Different Sources**




Source: Karanis et al., 1992.

**Figure 8-6. Impact of Growth Stage of *A. rhysodes* on the Required UV Dosage to Achieve Inactivation**

Results from recent studies show a potential for inactivating *Cryptosporidium parvum* oocysts using ultraviolet light disinfection. A 2 to 3-log reduction in the viability of *Cryptosporidium parvum* oocysts was achieved using a low-pressure ultraviolet light system with a theoretical minimum intensity of 14.58 mW/cm$^2$ and a contact time of 10 minutes (ultraviolet dose of 8,748 mW·s/cm$^2$) (Campbell et al., 1995). The combination filter-UV system described by Johnson (1997) is capable of delivering doses as high as 8,000 mW·s/cm$^2$, sufficient to achieve 2-log *Cryptosporidium* oocyst inactivation.

A pulsed UV process that delivered a minimum dose of 1,900 mW·s/cm$^2$ to any particle within the reactor was found to achieve *Cryptosporidium* oocyst inactivation levels in the range of 2-log. (Clancy et al., 1997). In this study, the reactor residence time was 4.7 seconds and the unit was operated to deliver 46.5 pulses per volume (10 Hz). Each pulse transfers power at the intensity rate of about 41 mW·s/cm$^2$ .

#### *8.4.3.3 UV Dose Summary*

UV radiation is considered effective for inactivating bacterial and viral pathogens. UV doses for 2 and 3-log inactivation of viruses are 21 and 36 mW·s/cm$^2$, respectively (AWWA, 1991). These doses are based on studies of hepatitis A virus inactivation and were derived by applying a safety factor of 3 to the inactivation data. Results from a more recent study on several ground water sources of hepatitis A virus, indicate that a 6 to 15 mW·s/cm$^2$ dose is required for a 4-log inactivation (Snicer et al., 1996). A safety factor was not applied to the doses. A 4-log inactivation of bacteriophage MS-2 is achieved at a dose of 93 mW·s/cm$^2$ (Snicer et al., 1996). In ground water containing high iron levels (0.65 ppm), applying a safety factor of 1.5 to the highest reported dose against viruses will yield a UV dose of 140 mWs/cm$^2$ for a minimum 4-log inactivation of viruses

In summary, ultraviolet radiation is effective against bacteria and viruses at low dosages. However, much higher dosages are required for *Cryptosporidium* and *Giardia* inactivation.

## 8.5 Disinfection Byproducts of UV Radiation

Unlike other disinfectants, UV does not inactivate microorganisms by chemical reaction. However, UV radiation causes a photochemical reaction in the organism RNA and DNA. The literature suggests that UV radiation of water can result in the formation of ozone or radical oxidants (Ellis and Wells, 1941; Murov, 1973). Because of this reaction, there is interest in determining whether UV forms similar byproducts to those formed by ozonation or advanced oxidation processes.

### 8.5.1 Ground Water

Malley et al. (1995) analyzed 20 ground water samples for aldehydes and ketones before and after UV radiation. Only one ground water sample, which contained 24 mg/L non-purgeable DOC and was highly colored, contained DBPs after exposure to UV. Low levels of formaldehyde were measure in duplicated experiments for this UV treated ground water sample mentioned above. GC-

ECD chromatographs before and after UV radiation for the other 19 ground waters studied showed significant shifts or unknown peaks after exposure to UV.

Malley et al. (1995) also determined the influence of UV on DBP formation during subsequent chlorination. To examine these effects, the 20 ground water samples were subjected to simulated distribution system (SDS) DBP tests with chlorine, before and after UV radiation. The data indicate that UV radiation did not significantly alter the SDS/DBP formation by chlorine in the ground waters studied.

To examine the effects of varying UV dosages on DBP formation, six new ground water samples (Malley et al., 1995) were subjected to UV dosages of 60, 130, and 200 $mW \cdot s/cm^2$. In this case, DBPs were not formed by UV radiation for any of the ground waters tested at any of the UV dosages. A comparison of chromatographs for samples before and after UV radiation, and for each UV dosage, showed no significant differences or appearances of unknown peaks.

## 8.5.2 Surface Water

UV radiation was found to produce low levels of formaldehyde in the majority of surface waters studied (Malley et al., 1995). The highest formaldehyde concentrations, ranging up to 14 μg/L, were observed in UV treatment of raw water, whereas trace levels (1 to 2 μg/L) were found in UV treatment of conventionally treated water. Since formaldehyde formation was also observed for one of the ground water samples, it appears that UV radiation of waters containing humic matter (i.e., color producing, UV absorbing organic macromolecules) will result in low levels of formaldehyde formation. Chromatographic examination of the surface water samples before and after UV radiation showed no other significant changes in the GC-ECD chromatograms.

Because of the chlorine demands of surface waters, higher chlorine dosages were required for post disinfection following UV radiation. This resulted in larger DBP concentrations than in the ground waters studied (Malley et al., 1995). However, the overall effect of UV radiation on SDS/DBPs was insignificant. As in the ground water studies, UV radiation did not significantly alter the total concentration or the speciation of the disinfection byproducts (e.g., THMs, HAA5, HANs, or HKs).

## 8.5.3 DBP Formation with Chlorination and Chloramination following UV Radiation

Research results suggest that UV radiation does not directly form DBPs or alter the concentration or species of DBPs formed by post-disinfection (Malley et al., 1995). However, the question of whether UV radiation influences the rate of DBP formation by post-disinfection is important. Several studies have addressed this question. Two surface waters that produced significant concentrations of a wide variety of DBPs in previous tests were chosen as samples. With the chlorine residuals carefully monitored to ensure they were consistent for pre-UV and post-UV samples, the results of the experiments suggested that UV radiation did not significantly affect the rate of DBP formation.

Studies were only performed to determine the pentane extractable DBP formation rate of a surface water sample for varying pH conditions. The results showed that UV radiation did not affect the rate of chloroform formation at pH 8.0 (Malley et al., 1995). Similarly, UV did not affect the DBP formation rate at pH 5.0. At pH 8.0, chloroform was the only pentane extractable detected, whereas at pH 5.0, chloroform, bromodichloromethane, chlorodibromomethane, and 1,1,1-trichloroacetone were formed.

The effects of UV radiation on the DBP formation rate following chloramination were also tested in this study using a surface water sample (Malley et al., 1995). Chloroform, trichloroacetic acid, dichloroacetonitrile (at low levels), and cyanogen chloride (at low levels) were the only detectable DBPs. Chloroform was the only compound formed at pH 8.0, and its rate of formation was not affected by UV radiation. At pH 5.0, chloroform and dichloroacetonitrile were formed, but their rate of formation was unaffected by UV radiation. Data showed that the effects of UV radiation on cyanogen chloride formation at pH 8.0 and pH 5.0 had no significant trends.

In summary, the DBP formation rate studies indicated that UV radiation did not significantly affect DBP formation rates when chlorine or chloramines were used as the post-disinfectant.

## 8.6 Status of Analytical Methods

Ultraviolet radiation intensity meter readings are taken to monitor the output of the UV disinfection system. These readings, coupled with the flow through the UV reactor, are used to determine the UV dosage applied. UV radiation leaves no residual disinfectant behind to monitor. Therefore, some form of secondary chemical disinfectant should be added to protect the distribution system against coliform proliferation and biofilm formation. Analytical methods for these chemical disinfectants are discussed elsewhere in this report.

### 8.6.1 Monitoring of Generated Ultraviolet Radiation

Ultraviolet intensity at 253.7 nm (the predominant wavelength emitted by low pressure mercury vapor lamps) is the water quality parameter used to monitor UV disinfection system output (Snider et al., 1991). The rate of disinfection is directly related to the average intensity of the UV light. Since UV intensity probes can only indicate UV intensity at a single point, there is no practical way to measure the average intensity of a UV system in the field by the operator.

The average intensity is dependent upon the three dimensional lamp geometry. Scheible (1983) developed a mathematical model that calculates the intensity at any point within the UV reactor. This Point Source Summation (PSS) method is used to estimate the average intensity emitted by any specific unit. UV disinfection system manufacturers use this method to design the system. The single point UV intensity probe reading is typically used for routine monitoring of the UV system only.

UV intensity sensors are typically photodiode sensors properly filtered to monitor the lamp intensity in the germicidal range only (DeMers and Renner, 1992). A minimum of two sensors (mounted near

separate lamps) located at the center of each lamp for each reactor is recommended as part of the system controls and instrumentation. White (1992) recommends installing the sensors in the wall of the disinfection chamber at the point of greatest distance from the tube or tubes.

The UV sensors should continuously sense the UV intensity produced in the bank of lamps. The sensor should be field calibrated to account for lamp geometry. Each UV sensor installed should provide a "low UV output" warning and a "low-low UV output" alarm indication that is field adjustable.

#### *8.6.1.1 Ultraviolet Sensor Performance*

To test electronic sensor performance, Snicer et al. (1996) placed a single electronic UV sensor at the center of a UV reactor. The UV sensor converted the UV energy into an electronic signal, which was used to indicate system performance. Initial results indicated that the sensor originally installed with the two UV reactors tended to wear or degrade in performance over time. The original sensor readings in the pilot facility were erratic and had a general downward trend over 6 months of operation. This loss in performance could not be attributed to UV lamp aging. In addition, these readings did not correlate well to actual system performance. After operation of these original sensors for 6 months, the manufacturer was consulted and a new type of sensor was installed into both pilot facilities. The new sensor design specifically addressed the problems encountered during the initial 6 months of the study. These new proprietary sensors performed consistently and sensor readings were shown to correlate well with actual MS-2 coliphage inactivation.

### 8.6.2 Disinfectant Interferences

Suspended solids may be the most important water quality parameter impacting UV intensity measurements. Particles can harbor bacteria and at least partially protect them from UV light. Particles can be completely penetrated, partially penetrated, or scatter UV light (Figure 8-4). All particles in water may not absorb UV light. Qualls et al. (1983) suggested that clays merely scatter UV light and, therefore, do little to inhibit performance.

Yip and Konasewich (1972) listed many chemical substances that interfere with UV transmission at 253.7 nm including phenolic compounds, humic acids, and ferric iron.

## 8.7 Operational Considerations

Onsite pilot plant testing is recommended to determine the efficiency and adequacy of UV disinfection for a specific quality of water. The efficiency test involves injecting select microorganisms into influent water and sampling effluent water to determine survival rates. The National Science Foundation's Standard 55 for ultraviolet water treatment systems recommends that UV disinfection systems not be used if the UV transmittance is less than 75 percent (NSF, 1991). If the raw water UV transmittance is less that 75 percent, the UV system should be proceeded by other treatment processes (to increase UV transmittance) or a different disinfectant should be used.

As previously discussed, some constituents that adversely interfere with UV disinfection performance by either scattering and/or absorbing radiation are iron, chromium, copper, cobalt, sulfites, and nitrites. Care should be taken with chemical processes upstream of UV disinfection process to minimize increasing concentrations of these constituents since disinfection efficiency may be adversely affected.

## 8.7.1 Equipment Operation

UV disinfection facilities should be designed to provide flexibility in handling varying flow rates. For lower flow rates, a single reactor vessel should be capable of handling the entire flow rate. A second reactor vessel with equal capacity of the first reactor vessel should be provided for redundancy should the first reactor vessel be taken out of service. For higher flow rates, multiple reactor vessels should be provided with lead/lag operation and flow split capacity to balance run time for each reactor vessel, if desired, and to avoid hydraulic overloading. Valves should be provided within the interconnecting piping to isolate one reactor vessel from another. There should also be a positive drainage system to remove water from within a reactor vessel when it is taken out of service.

### *8.7.1.1 UV Lamp Aging*

The output of UV lamps diminishes with time. Two factors that affect their performance are: solarization which is the effect UV radiation has on the UV lamp that causes it to become opaque; and, electrode failure which occurs when electrodes deteriorate progressively each time the UV lamp is cycled on and off. Frequent lamp cycling will lead to premature lamp aging. When determining the requirement for UV disinfection, a 30 percent reduction of UV output should be used to estimate end of lamp. Average life expectancy for low pressure UV lamps is approximately 8,800 hours.

### *8.7.1.2 Quartz Sleeve Fouling*

Fouling of the quartz sleeve reduces the amount of UV radiation reaching the water. The quartz sleeve has a transmissibility of over 90 percent when new and clean. Over time, the surface of the quartz sleeve that is in contact with the water starts collecting organic and inorganic debris (e.g., iron, calcium, silt) causing a reduction in transmissibility (USEPA, 1996). When determining the requirements for UV disinfection, a 30 percent reduction of UV transmission should be used to reflect the effect of quartz sleeve fouling.

## 8.7.2 Equipment Maintenance

### *8.7.2.1 UV Lamp Replacement*

Adequate space should be provided around the perimeter of the reactor vessels to allow access for maintenance and replacement of UV lamps. With modular electrical fittings, lamp replacement consists of unplugging the pronged connection of the old lamp and plugging in the new.

#### 8.7.2.2 Quartz Sleeve Cleaning

Quartz sleeve cleaning may be accomplished by physical or chemical means. Physical alternatives include:

- Automatic mechanical wiper;
- Ultrasonic devices;
- High water pressure wash; and
- Air scour.

Chemical cleaning agents include sulfuric or hydrochloric acid. A UV reactor vessel may contain one or more physical cleaning system with provision for an occasional chemical cleaning.

#### 8.7.2.3 Miscellaneous

Effective maintenance of a UV system will involve:

- Periodic checks for proper operation;
- Calibration of intensity meter for proper sensitivity; and
- Inspect and/or clean reactor vessel interior.

### 8.7.3 Standby Power

Producing UV radiation requires electricity to power the electronic ballasts, which in turn power the UV lamps. Since disinfection is of utmost importance in producing potable water, the UV system should remain in service during periods of primary power failure. A dual power feed system or essential circuitry powered by a standby generator are typical ways to achieve the desired reliability. Each low pressure UV lamp requires approximately 100 Watts of standby power. A second precaution that should be considered is not powering the UV system from the same motor control center (MCC) that powers variable frequency drives (VFDs). The electronic ballasts produce harmonics that may require mitigation (active harmonic filters) for the VFDs.

## 8.8 Summary Table

**Table 8-3. Summary of UV Disinfection**

| Consideration | Description |
|---|---|
| Generation | Low pressure and medium pressure UV lamps available. |
| Primary uses | Primary physical disinfectant; requires secondary chemical disinfectant for residual in distribution system. |
| Inactivation efficiency | Very effective against bacteria and viruses at low dosages (5-25 mW•s/cm$^2$ for 2-log removal and 90-140 mW•s/cm$^2$ for 4-log removal). Much higher dosage required for *Cryptosporidium* and *Giardia* (100-8,000 mW•s/cm$^2$ for 2-log removal). |
| Byproduct formation | Minimal disinfection byproducts produced. |
| Limitations | Limited experience and data with flows greater than 200 GPM. Water with high concentrations of iron, calcium, turbidity, and phenols may not be applicable to UV disinfection. |
| Point of application | Prior to distribution system. |
| Special considerations | Extremely high UV dosages for *Cryptosporidium* and *Giardia* may make surface water treatment impractical. |

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# 9. COMBINED DISINFECTANTS

Multiple disinfectants, the sequential or simultaneous use of two or more disinfectants, have been used with increasing frequency in recent years. This trend is attributed to the fact that:

- Less reactive disinfectants, such as chloramines, have proven to be quite effective in reducing DBPs formed during disinfection and are more effective for controlling biofilms in the distribution system.
- Regulatory and consumer pressure to produce water that has been disinfected to achieve high inactivation for various pathogens, has pushed the industry towards more effective disinfectants. Sometimes more effective disinfection meant using higher disinfectant doses which also produces more DBPs.
- Recent research has shown that the application of sequential disinfectants is more effective than the added effect of the individual disinfectants. This process where two (or more) disinfectants produce a synergistic effect by either simultaneous or sequential application to achieve more effective pathogen inactivation, is referred to as ***interactive disinfection*** in this manual.

This chapter discusses recent industry applications of multiple disinfectants to meet the varied requirements for inactivation and reduction in DBPs. The initial discussion focuses on traditional disinfectants used in primary and secondary application. This is followed by a discussion of interactive disinfectants where two disinfectants are applied together specifically to achieve primary disinfection. Note that the IESWTR does not have any provision for additional credits for interactive disinfection or taking additional credit for the synergistic effects from interactive disinfection. Until such credit is established, interactive disinfection is considered an emerging technology. This chapter does not discuss mixed oxidant systems, which are designed to generate mixed oxidants on-site for drinking water disinfection, and are also considered an emerging technology.

## 9.1 Primary and Secondary Disinfectants

By separating the inactivation function and residual disinfection function in water treatment, each can be optimized independently. Thus, the combination of disinfectants currently used in disinfection is typically identified as a primary or secondary disinfectant, as follows:

- Primary disinfection refers to the inactivation of microorganisms to meet the regulatory bacteriological requirements. This requirement typically is met by achieving certain CT requirements to assure a target log inactivation of target organisms as set forth in the Surface Water Treatment Rule (SWTR) (AWWA, 1991).
- Secondary disinfection refers to application of a disinfectant to meet regulatory requirements for distribution system bacteriological quality as set forth in the Total Coliform Rule (TCR). The SWTR requires that a residual disinfectant be measured in the distribution system, or that the bacteriological quality meet certain standards (heterotrophic plate count (HPC) less than 500/100mL).

To be an effective primary disinfectant, the disinfectant should effectively inactivate the target organism. Table 9-1 lists potential primary disinfectants as discussed in Chapters 2 through 8.

**Table 9-1. Potential Primary Disinfectants**

| Target Organism | Potential Primary Disinfectants | |
|---|---|---|
| | With Filtration[1] | Without Filtration |
| Coliform Bacteria | Chlorine<br>Chloramines<br>Chlorine dioxide<br>Ozone<br>UV<br>Interactive disinfection | Chlorine<br>Chlorine dioxide<br>Interactive disinfection[3] |
| Giardia cysts | Chlorine[2]<br>Chlorine dioxide[2]<br>Ozone[2]<br>Interactive disinfection | Chlorine[2]<br>Chlorine dioxide[2]<br>Interactive disinfection[3] |
| Viruses | Chlorine[2]<br>Chlorine dioxide[2]<br>Ozone[2]<br>UV[2]<br>Interactive disinfection | Chlorine[2]<br>Chlorine dioxide[2]<br>UV[2]<br>Interactive disinfection[3] |
| Cryptosporidium oocysts | Chlorine dioxide<br>Ozone<br>Interactive disinfection | Chlorine dioxide<br>Interactive disinfection[3] |

[1] Natural or treatment filtration reduces disinfection inactivation requirements.

[2] Inactivation credit established in SWTR.

[3] Any interactive disinfection that uses ozone or peroxone without filtration is strongly discouraged.

As discussed in earlier chapters, certain disinfectants (e.g., ozone, UV, peroxone, and in some cases chlorine dioxide), while being effective disinfectants, do not leave a long-lasting residual disinfectant. Therefore, secondary disinfection is limited to those disinfectants that remain stable in the distribution system. In order of decreasing stability, these secondary disinfectants are chloramines, chlorine, and chlorine dioxide.

Based on the above, the combinations of disinfectants that are viable options to meet the disinfection requirements can be determined for various treatment trains. These combinations are shown for various treatment objectives. Note that the treatment objectives are dependent on the treatment currently in place.

To meet DBP, and specifically, THM limits, several studies have evaluated the application of various primary/secondary disinfectants. Table 9-2 presents the typical application of these combined disinfectants.

**Table 9-2. Primary/Secondary Disinfectant Combinations and Typical Applications in Water Treatment**

| Primary / Secondary | Typical application* | Comment |
|---|---|---|
| Chlorine/Chlorine | Low THMFP raw water, low TOC, conventional treatment with optimal coagulation. | Most commonly used disinfection scheme. Effective system. |
| Chlorine/Chloramine | Moderate THM production situation, typically with conventional treatment. | Chlorine to provide disinfection and monochloramine to limit DBP formation. |
| Chlorine dioxide/Chlorine dioxide | High DBP production, require filter process to remove *Cryptosporidium*, low chlorine dioxide demand in treated water. | Primary and secondary usage requires a limit on chlorine dioxide dose to reduce residual chlorate/chlorite. |
| Chlorine dioxide/Chloramine | High DBP production, require filtration to remove *Cryptosporidium*. | Primary chlorine dioxide dose limited to residual chlorate/chlorite. Stable, low reactive secondary disinfectant. |
| Ozone/Chlorine | Moderate DBP formation, direct or no filtration, low THMFP. | Highly effective disinfection to achieve high log inactivation; low THMFP to accept free chlorine. |
| Ozone/Chloramine | Moderate DBP formation, direct or no filtration, higher THMFP. | Highly effective disinfection to achieve high log inactivation, low THMFP to require combined chlorine residual. |
| UV/Chlorine | Requires membrane treatment to provide effective *Giardia* and *Cryptosporidium* removal. UV only for virus inactivation; ground water disinfection; low THMFP. | Rare application but feasible in special circumstances. Little *Giardia* and no *Cryptosporidium* inactivation. |
| UV/Chloramine | Requires membrane treatment to provide effective *Giardia* and *Cryptosporidium* removal. UV only for virus inactivation; ground water disinfection, moderate THMFP. | Rare application but feasible in special circumstances. No *Giardia* or *Cryptosporidium* inactivation. |

Notes:

* Low DBP formation is defined as producing less than the Stage 2 D/DBP Proposed Standard (less than 0.040 mg/L TTHM; less than 0.030 mg/L HAA5). Moderate DBP formation is defined as producing less than the Stage 1 D/DBP Standard and more than the Stage 2 D/DBP Proposed Standard. High DBP formation is defined as producing more than the Stage 1 D/DBP Standard (greater than 0.080 mg/L TTHM; greater than 0.060 mg/L HAA5).

## 9.1.1 DBP Formation with Various Primary and Secondary Disinfectant Combinations

The concentrations and types of DBPs formed depend on, among other things, the combination of disinfectants used to achieve primary and secondary disinfection and the water quality. For example, under certain water quality conditions, ozone/chloramine disinfection is known to produce lower THM concentrations than chlorine/chloramine disinfection. However, the ozone/chloramine alternative can increase the formation of other DBPs such as aldehydes and BOM. No single

combination of disinfectants is applicable to all situations. Table 9-3 summarizes the potential DBPs formed by various combinations of disinfectants. The disinfection byproducts referenced here are discussed in greater detail in earlier chapters of this manual.

## Table 9-3. DBPs Associated with Various Combined Oxidation/Disinfection Processes

| Alternative | | | Potential DBPs | Remarks |
|---|---|---|---|---|
| **Preoxidation** | **Primary Disinfection** | **Secondary Disinfection** | | |
| Chlorine | Chlorine | Chlorine | XDBPs* | Maximum XDBP formation compared to all other strategies. Principal components are TTHMs and HAAs. |
| | | | Aldehydes | Formed at relatively low levels. |
| Chlorine | Chlorine | Chloramine | XDBPs<br>Cyanogen chloride<br>Cyanogen bromide | Formation of XDBPs (specifically TTHMs and HAA5s) significantly reduced compared to chlorine/ chlorine/ chlorine. |
| | | | Aldehydes | Formed at relatively low levels. |
| Chlorine dioxide | Chlorine dioxide | Chlorine | XDBPs | Formation of XDBPs may be decreased by delaying the point of chlorine addition. |
| | | | Aldehydes, carboxylic acids, maleic acids | Formed at relatively low levels. |
| | | | Chlorate<br>Chlorite | Chlorite is a major breakdown product of chlorine dioxide. |
| Chlorine dioxide | Chlorine dioxide | Chloramine | XDBPs | Formation of XDBPs (especially TTHMs and HAA5s) minimized by avoiding use of free chlorine. |
| | | | Aldehydes, carboxylic acids, maleic acid | Formed at relatively low levels. |
| | | | Chlorate<br>Chlorite | Chlorite is a major breakdown product of chlorine dioxide. |
| Potassium permanganate | Chlorine | Chlorine | XDBPs | Formation of XDBPs may be decreased by delaying the point of chlorine addition. |
| | | | Aldehydes | Formed at relatively low levels. |
| Potassium permanganate | Chlorine | Chloramine | XDBPs<br>Cyanogen chloride<br>Cyanogen bromide | Formation of XDBPs may further be decreased compared to potassium permanganate/ chlorine/ chlorine. |
| | | | Aldehydes | Formed at relatively low levels. |
| Ozone | Ozone | Chlorine | XDBPs | Formation of certain XDBPs may increase or decrease compared to chlorine/ chlorine/ chlorine. Brominated byproducts may be of concern when bromides are present in the raw water. |
| | | | Bromate, Aldehydes, carboxylic acids | Although formed at relatively high levels significant amounts of this BOM can be removed through biological filtration. |
| Ozone | Ozone | Chloramine | XDBPs<br>Cyanogen chloride<br><br>Cyanogen bromide | Formation of XDBPs (especially TTHMs) minimized by avoiding use of free chlorine. |
| | | | Bromate, Aldehydes, carboxylic acids | Although formed at relatively high levels significant amounts of this BOM can be removed through biological filtration. |

| Alternative | | | Potential DBPs | Remarks |
|---|---|---|---|---|
| **Preoxidation** | **Primary Disinfection** | **Secondary Disinfection** | | |
| Peroxone | Chlorine or Ozone | Chlorine | XDBPs | Formation of certain XDBPs may increase or decrease compared to chlorine/ chlorine/ chlorine. |
| | | | Bromate, Aldehydes, carboxylic acids | Although formed at relatively high levels significant amounts of this BOM can be removed through biological filtration. Also, the formation of bromate will increase if peroxone is used. |
| Peroxone | Chlorine or Ozone | Chloramine | XDBPs<br>Cyanogen chloride<br>Cyanogen bromide | Formation of XDBPs may decrease compared to peroxone/ chlorine/ chlorine. |
| | | | Bromate, Aldehydes, carboxylic acids | Although formed at relatively high levels significant amounts of this BOM can be removed through biological filtration. Also, the formation of bromate will increase if peroxone is used. |
| Chlorine | UV** | Chloramine | XDBPs<br>Cyanogen chloride<br>Cyanogen bromide | May form XDBP from pre-oxidation. |
| | | | Aldehydes | Low levels. |
| Potassium Permanganate | UV** | Chloramine | XDBPs | Very low due to less reactive oxidants. |
| | | | Aldehydes, carboxylic acids | Very low, if any, due to less reactive oxidants. |

* XDBPs - Halogenated Disinfection Byproducts.

* * Although "conventional" UV use as primary disinfectant is limited to virus inactivation (may require membrane filtration), pulsed UV may be able to inactivate *Giardia* and *Cryptosporidium*.

Source: Adopted in part from USEPA, 1992; Richardson et al., 1994.

Raw water chlorination, applied prior to natural organic matter (NOM) removal processes, combined with chlorination for residual disinfection produces the greatest concentrations of halogenated DBPs. Studies indicate that pre-oxidation of raw water with ozone or chlorine dioxide can reduce the formation of halogenated DBPs because it shifts the point of chlorine application from raw water to settled or filtered water which has lower DBP precursor concentrations (MWDSC and JMM, 1989).

The use of ozone can reduce the formation of halogenated byproducts in waters containing low concentrations of bromide. However, ozone increases BOM and may encourage bacterial growth in the distribution system. Removal of AOC with biological filtration (e.g., biological activated carbon) reduces the potential for bacterial growth in the distribution system. The use of chloramines as a secondary disinfectant instead of chlorine shortens the chlorine contact time and thus reduces the formation of chlorinated byproducts. However, chloramine produces by-products of its own (cyanogen chloride and cyanogen bromide). In addition, a short chlorine contact time prior to ammonia addition will help inactivate heterotrophic plate count bacteria that are found in the effluent of a biologically active filter. Bench or pilot studies will be required to evaluate the trade-offs in DBP formation for various disinfectant combinations for a specific application.

The application of ozone should be carefully considered because it produces aldehydes, aldoketoacids, and carboxylic acids. However, these can be removed in a biologically active filter.

In bromide-containing waters, ozonation can increase the formation of brominated organic DBPs and form bromate.

In pilot plant studies for water containing low concentrations of bromide, Lykins et al. (1991) determined that ozonation followed by chloramination produced the lowest levels of halogenated disinfection byproducts. However, this is not applicable to source waters containing significant bromide concentrations due to the potential for bromate formation and brominated THMs and HAAs. The addition of chlorine dioxide will produce chlorite and chlorate and may form some oxygenated DBPs (e.g., maleic acids).

## 9.1.2 Impact of Modifying Disinfection Practices

EPA and the Association of Metropolitan Water Agencies funded a 2-year study of 35 water treatment facilities to evaluate DBP production. Among four of the facilities, eleven alternative disinfection strategies were instigated to evaluate the difference in DBP production from the plants' previously existing disinfection strategies. Three reports (MWDSC and JMM, 1989; Jacangelo, 1989; USEPA, 1992) analyzed and documented different aspects of the study. Table 9-4 shows the eleven potential strategies used for primary and secondary disinfection. Table 9-5 shows the changes in DBP production observed in the four plants after eight of these new strategies were implemented. Following are overviews of the potential implications of these strategies, as detailed by the three reports.

**Table 9-4. Strategies for Primary and Secondary Disinfectants**

| *Base Disinfection Condition* | *Modified Disinfection Practice* |
|---|---|
| Chlorine/Chlorine | Chlorine/Chloramine |
| Chlorine/Chlorine | Chloramine/Chloramine |
| Chlorine/Chlorine | Chlorine Dioxide/Chloramine |
| Chlorine/Chlorine | Ozone/Chlorine |
| Chlorine/Chlorine | Ozone/Chloramine |
| Chlorine/Chlorine | Chlorine Dioxide/Chlorine |
| Chlorine/Chlorine | Chlorine Dioxide/Chlorine Dioxide |
| Chlorine/Chloramine | Ozone/Chloramine |
| Chlorine/Chloramine | Chlorine Dioxide/Chloramine |
| Ozone/Chlorine | Ozone/Chloramine |
| Chloramine/Chloramine | Ozone/Chloramine |

Note: Disinfectants are listed as primary disinfectant/secondary disinfectant

## Table 9-5. Impacts of Disinfection Practice on DBP Formation

| Disinfection Byproduct | Change in Disinfection Practice (Primary Disinfectant/Secondary Disinfectant) | | | | |
|---|---|---|---|---|---|
| | Chlorine/Chlorine to Chlorine/Chloramine | Chlorine/Chlorine to Ozone/Chlorine | | Chlorine/Chloramine to Ozone/Chloramine | Chlorine/Chlorine to Chloramines/Chloramines |
| | Utility #7 | Utility #19 | Utility #36 | Utility#7 | Utility #36 |
| Total Trihalomethanes | decrease | decrease | no change | decrease | decrease |
| Total Haloacetic Acids | decrease | decrease | no change | decrease | decrease |
| Total Haloacetonitriles | decrease | decrease | no change | decrease | decrease |
| Total Haloketones | decrease | no change | increase | increase | decrease |
| Total Aldehydes | not analyzed | not analyzed | increase | not analyzed | decrease |
| Chloropicrin | no change | increase | increase | decrease | no change |
| Chloral Hydrate | decrease | increase | increase | decrease | decrease |
| Cyanogen Chloride | no change | not analyzed | no change | no change | increase |
| **Disinfection By-Product** | **Change in Disinfection Practice (Primary Disinfectant/Secondary Disinfectant)** | | | | |
| | Ozone/Chlorine to Ozone/Chloramine | Chloramines/Chloramines to Ozone/Chloramines | | Chlorine/Chlorine to Ozone/Chloramine | |
| | Utility #36 | Utility #25 | Utility #36 | Utility #7 | Utility #36 |
| Total Trihalomethanes | decrease | decrease | no change | decrease | decrease |
| Total Haloacetic Acids | decrease | decrease | no change | decrease | decrease |
| Total Haloacetonitriles | decrease | no change | no change | decrease | decrease |
| Total Haloketones | decrease | no change | increase | decrease | decrease |
| Total Aldehydes | decrease | increase | increase | not analyzed | increase |
| Chloropicrin | increase | increase | increase | decrease | increase |
| Chloral Hydrate | decrease | decrease | increase | decrease | decrease |
| Cyanogen Chloride | increase | increase | increase | no change | increase |

Notes: Results based on full scale evaluation at Utilities #19 and #25 and on pilot scale evaluations at Utilities #7 and #36.

Free chlorine contact time was 4 hours for Utility #7 during use of chlorine/chloramine strategy.

Systems must demonstrate efficacy of chloramines as a primary disinfectant if they are to be used as such.

Source: USEPA, 1992; Jacangelo, et al., 1989.

## 9.1.3 Chlorine/Chlorine to Chlorine/Chloramine

Chloramines as a secondary disinfectant shortens the free chlorine contact time and, therefore, reduce the formation of most halogenated DBPs, including TTHM. However, the consumer is exposed to a chloramine residual that has a different health concern (i.e., can cause methemoglobinemia and adversely affect the health of kidney dialysis patients) than chlorine residual. For maximum benefit in reducing halogenated byproducts, the ammonia/chlorine mixing conditions should be optimized to quickly remove free chlorine from the system, with the optimal pH for the chlorine/ammonia reaction considered to be pH 8.3. In one case, changing the secondary disinfectant from chlorine to chloramines resulted in a reduction in average TTHM concentrations from 105 μg/L to 45 μg/L with most (80 percent) of the TTHM formation in the chloraminated system occurring prior to the addition of ammonia (Singer, 1988).

## 9.1.4 Chlorine/Chlorine to Ozone/Chlorine

A change from chlorine to ozone for primary disinfection is attractive because ozone generally produces fewer regulated DBPs than chlorine. However, as mentioned in previous chapters, ozone will react with organic compounds to produce BOM, aldehydes, organic acids, and ketones, many of which are biodegradable. In addition, for water sources containing significant bromide ion concentrations, site-specific engineering evaluations should be completed prior to using ozone as a primary disinfectant due to the potential formation of bromate ion or brominated organic byproducts. At low pH values, the brominated organic byproducts are favored, while at higher pH values, bromate ion formation is favored. Therefore, changing from chlorine to ozone may be limited in cases where the source water contains bromide ion. In the ozone/chlorine strategy, bromine has the opportunity to react with NOM prior to chlorine application, potentially increasing the extent of bromine incorporation into DBPs. Also, in this situation, switching to biological filtration can increase regrowth and corrosion rates in the distribution system.

In systems containing negligible concentrations of bromide ion, replacement of pre-chlorination with ozonation may allow for the point of chlorine addition to be moved beyond clarification or filtration. The principal benefit derived from using ozone for controlling THM formation is that it allows free chlorine to be applied later in the treatment process after precursors have been removed. This would be expected to result in reduced formation of chlorinated DBPs due to the reduced chlorine dose requirement and the reduced NOM concentrations following treatment. Replacement of chlorine with ozone appears to be most effective in decreasing chlorinated DBPs when it eliminates the need for pre-chlorination. However, replacing chlorine with ozone while maintaining free chlorine as a secondary disinfectant has produced mixed results, with some byproduct concentrations being reduced at the expense of increasing the concentrations of others. *The human health implications of this trade-off are largely unknown.* It has been shown that substituting ozone for chlorine as a primary disinfectant, while maintaining free chlorine as a secondary disinfectant when not practicing pre-chlorination, resulted in an increased THM formation in raw waters for the Cities of Phoenix and San Diego (Malcolm Pirnie, 1989; Malcolm Pirnie, 1990).

An operational consideration of the ozone/chlorine system is the application point for chlorine. The ozone should be completely decomposed or chemically quenched prior to chlorine addition. If ozone is present when chlorine is added, the ozone will react with the chlorine and NOM present to form chlorinated DBPs.

Ozonation converts NOM into low molecular weight humic NOM and may increase the concentrations of precursors to some DBPs. For instance, ozonation followed by chlorination as a secondary disinfectant may yield high concentrations of chloral hydrate (Logsdon et al., 1992; McKnight and Reckhow, 1992). This may occur because the byproduct of ozonation, acetaldehyde, is a known precursor for chloral hydrate, a byproduct of chlorination. Enhancement of chloral hydrate has not been observed when biologically active filtration is used following ozonation and prior to chlorination (Singer, 1992). In addition to chloral hydrate, ozonation followed by chlorination can produce greater THM and haloketones levels than chlorination alone, particularly when chlorine is applied at high pH levels (Jacangelo et al., 1989; Reckhow et al., 1986). Ozonation followed by chlorination or chloramination can increase chloropicrin and cyanogen chloride concentrations above those observed with chlorination or chloramination alone (Jacangelo et al., 1989). The most promising treatment strategy for preventing the enhancement of these biodegradable ozonation byproducts and BOM is to locate ozonation after sedimentation and follow it by biologically active GAC.

## 9.1.5 Chlorine/Chlorine to Ozone/Chloramine

In addition to the concerns addressed in Sections 9.1.3 and 9.1.4, switching from chlorine to chloramine residual exposes the consumer to a residual that may be a more significant health concern (particularly for kidney dialysis patients). The impact of switching from chlorine/chlorine to ozone/chloramines on the production of byproducts was investigated in a 5 gpm pilot study (MWDSC and JMM, 1989; Jacangelo et al., 1989). That switch produced greater concentrations of chloropicrin, cyanogen chloride, formaldehyde and total aldehydes than in the original chlorine/chlorine strategy. Concentrations of TTHMs, total haloacetic acids, total haloacetonitriles, total haloketones and chloral hydrate were lower with ozone/chloramine. Brominated DBPs were not reported. Ozonation followed by chloramination has been observed to increase cyanogen chloride concentrations beyond those observed with chlorination only (Jacangelo et al., 1989). Increased chloral hydrate has not been observed when monochloramine is applied as the secondary disinfectant (Singer, 1992).

## 9.1.6 Chlorine/Chlorine to Chlorine Dioxide/Chlorine

Use of chlorine dioxide as a pre-oxidant to replace chlorine may allow moving the point of chlorination downstream in the process train for application to water with lower NOM concentrations. The reduced precursor concentration and the reduced chlorine dose should result in a reduction of chlorinated DBPs. However, if excess chlorine is present in the chlorine dioxide feed stream, it would react with NOM prior to removal in sedimentation and filtration if pre-oxidation is practiced. Switching from chlorine/chlorine to chlorine dioxide/chlorine produces mixed results.

Like ozone, chlorine dioxide alters the nature of NOM molecules, potentially forming greater precursor concentrations for some DBPs while reducing the precursor concentrations for other DBPs. *The human health implications of these trade-offs are largely unknown.* Chlorine dioxide/chlorine appears to be most effective in decreasing chlorinated DBPs when it can replace the need for pre-chlorination. However, for facilities that use pre-chlorination but do not require it, continuing to use chlorine/chlorine while eliminating pre-chlorination may be as effective in decreasing chlorinated DBPs.

## 9.1.7 Chlorine/Chlorine to Chlorine Dioxide/Chlorine Dioxide

The potential to apply chlorine dioxide as both a primary and secondary disinfectant is limited because:

- Chlorine dioxide is a strong oxidant and dissipates rapidly in both raw and treated waters; and
- Approximately 50 to 70 percent of chlorine dioxide is converted to the inorganic byproducts chlorite and chlorate.

On the positive side, chlorine dioxide/chlorine dioxide application will significantly lower the formation of organic DBPs.

## 9.1.8 Chlorine/Chloramine to Ozone/Chloramine

In addition to the concerns raised in Sections 9.1.4 and 9.1.5, switching from chlorine/chloramine to ozone/chloramine resulted in reduced formation of most of the halogenated DBPs (MWDSC and JMM, 1989). Other studies also indicate reduction in the formation of most halogenated DBPs but increased formation of 1,1-dichloropropanone (MWDSC and JMM, 1989). The primary difference in chlorinated DBP formation when switching from chlorine/chloramine to ozone/chloramine could be attributed to the shorter contact time with free chlorine.

## 9.1.9 Chlorine/Chloramine to Chlorine Dioxide/Chloramine

The Louisville Water Company evaluated the feasibility of switching from a chlorine/chloramine to chlorine dioxide followed by chloramine to control THM formation (Hubbs et al., 1981). The treatment plant includes lime soda-ash for softening. Disinfection occurs prior to the lime treatment step. Ammonia is added to form chloramine before the water enters the softening phase. There is a 10 minute lag period between the first disinfectant (chlorine or chlorine dioxide) and second disinfectant (chloramine) addition. The study showed a significant decrease in THM formation from 25 μg/L with chlorine to 5 μg/L using chlorine dioxide as the initial disinfectant. At the same time, treated water coliform densities were essentially unchanged; however, results showed slightly more scattered data during the chlorine dioxide test period. Based on these results, the Water Company decided to use chlorine/chloramine to meet disinfection and THM targets. No other DBPs were measured during the test period.

### 9.1.10 Ozone/Chlorine to Ozone/Chloramine

In addition to concerns raised in Sections 9.1.4 and 9.1.5, when compared with ozone/chlorine, ozone/chloramine produced greater concentrations of cyanogen chloride. Concentrations of TTHM, total haloacetic acids, total haloacetonitriles, total haloketones, chloral hydrate, total aldehydes and formaldehyde were lower with ozone/chloramine than with ozone/chlorine. Ozone/chloramine produces some chlorinated DBPs at greater concentrations than ozone/chlorine; however, ozone/chloramine significantly reduces TTHMs compared to ozone/chlorine (LeLacheur et al., 1991).

### 9.1.11 Summary

EPA is not encouraging systems to switch to different disinfectants due to unknown risks to public health. When needed for compliance with regulations or increasing *Cryptosporidium* inactivation, careful selection of alternative disinfectants as primary and secondary disinfectants, can produce less DBPs and increase inactivation. In general, the results followed the characteristic DBPs associated with the primary disinfectant (halogenated DBPs with chlorine compounds or ozone in the presence of bromide oxidized organics, AOC with ozone or peroxone). However, by carefully selecting the primary and secondary disinfectant and avoiding long contact times and high dosages of halogens (chlorine, bromine), the total DBP formation declined. The quantity and types of DBPs that form are site-specific, depending on the water quality, disinfectant dose and type, and are best determined by bench testing. Note that any system changing disinfectants is subject to the profiling and benchmarking requirements as described in Section 1.3.1 and specified in 40 CFR § 141.172.

## 9.2 Pathogen Inactivation with Interactive Disinfectants

In 1988, several reports appeared on the combined efficiency of some disinfectants on pathogen inactivation. Worley and Williams (1988) reported that a mixture of free chlorine and organic *N*-halamine produced higher levels of inactivation of a variety of bacteria. The combination of free chlorine and sodium bromide was also investigated and found to be more effective than using free chlorine alone (Alleman et al., 1988). In a study at the University of Arizona, the synergistic inactivation of *E. coli* and MS-2 coliphage was demonstrated by the combined application of chloramine and cupric chloride (Straub et al., 1994).

Recently there has been a great deal of interest in the potential of interactive disinfectants because reports showed that some of these combinations are more effective for inactivating *Cryptosporidium* (Finch et al., 1994). Research on interactive disinfectants for primary pathogen inactivation is under way for several combinations of disinfectants:

- Chlorine followed by chloramine;
- Chlorine dioxide followed by chlorine;
- Chlorine dioxide followed by chlorine dioxide;

- Chlorine dioxide followed by chloramine;
- Ozone followed by chlorine;
- Ozone followed by chlorine dioxide; and
- Ozone followed by chloramine.

## 9.2.1 Inactivation Mechanism

Bernbaum (1981 and 1985) developed a testing method for determining the kind of interaction that can be expected when agents are combined to produce a given observation. Synergism can be tested using the mathematical model developed by Bernbaum and modified for disinfection kinetics by Kouame and Haas (1991). The principle is that, if the agents in a given combination do not interact in producing the effect observed, then regardless of the effect relations, the following equation is satisfied:

$$\sum_{i=1}^{n} \frac{x_i}{y_i} = 1$$

where:

| | | |
|---|---|---|
| $x_i$ | = | Concentration of the individual agent in the combination |
| $y_i$ | = | Concentration of the agents that individually would produce the same magnitude of effect as that of the combination |
| $i$ | = | Individual agent |
| $n$ | = | Total number of agents |

The sum calculated from this equation for a set of data is interpreted as follows:

- The sum is less than 1 in the case of synergistic interaction;
- The sum is greater than 1 in the case of antagonistic interaction; and
- The sum is equal to 1 in the case of additivity (zero interaction).

Using this approach, Kouame and Haas (1991) showed that a synergistic interaction exists in the inactivation of *E. coli* when exposed simultaneously to free chlorine and monochloramine.

The authors described a possible mechanism in which both of the disinfectants work together to inactivate bacteria. The researchers hypothesized that bacterial inactivation is caused by monochloramine penetrating the cell and oxidizing thiol groups, which in turn causes structural changes in the cell membrane. Once these changes have been made, copper is allowed to pass into the cell and binds either to sulfhydryl groups of respiratory enzymes or nucleic acids. More recently, the researchers investigating *E. coli* inactivation hypothesized that a potential synergistic mechanism consisting of sub-lethal injury caused by free chlorine resulted in enhanced sensitivity to monochloramine (Kouame and Haas, 1991).

Another hypothesis for the increased effectiveness of interactive disinfectants is that the first oxidant (i.e., chlorine, chlorine dioxide, or ozone) conditions the outer membrane of *Cryptosporidium* oocysts so that the secondary oxidant (i.e., chlorine, chlorine dioxide, and monochloramine) can penetrate the oocyst more easily (Liyanage et al., 1996). For example, preliminary work on the disinfection of *Cryptosporidium parvum* using free chlorine followed by monochloramine suggested that there may be a synergism involving two chlorine species. Sequential treatment of these chlorine species was found to provide greater inactivation than expected from the additive effects of the two disinfectants used alone (Gyurek et al., 1996).

Recent studies have utilized a straight forward method to determine if synergism has occurred based on measured inactivation (Finch, 1997; Gyurek et al., 1996; and Liyanage et al., 1996). According to this approach, synergism is demonstrated if the sequential application of disinfectants provides more inactivation than is expected from the additive effects of the individual, separate disinfectants. In addition, the magnitude of the synergistic effects is equal to the difference in the level of inactivation achieved from multiple disinfectants and the additive inactivations achieved from the single disinfectants.

## 9.2.2 Environmental Effects

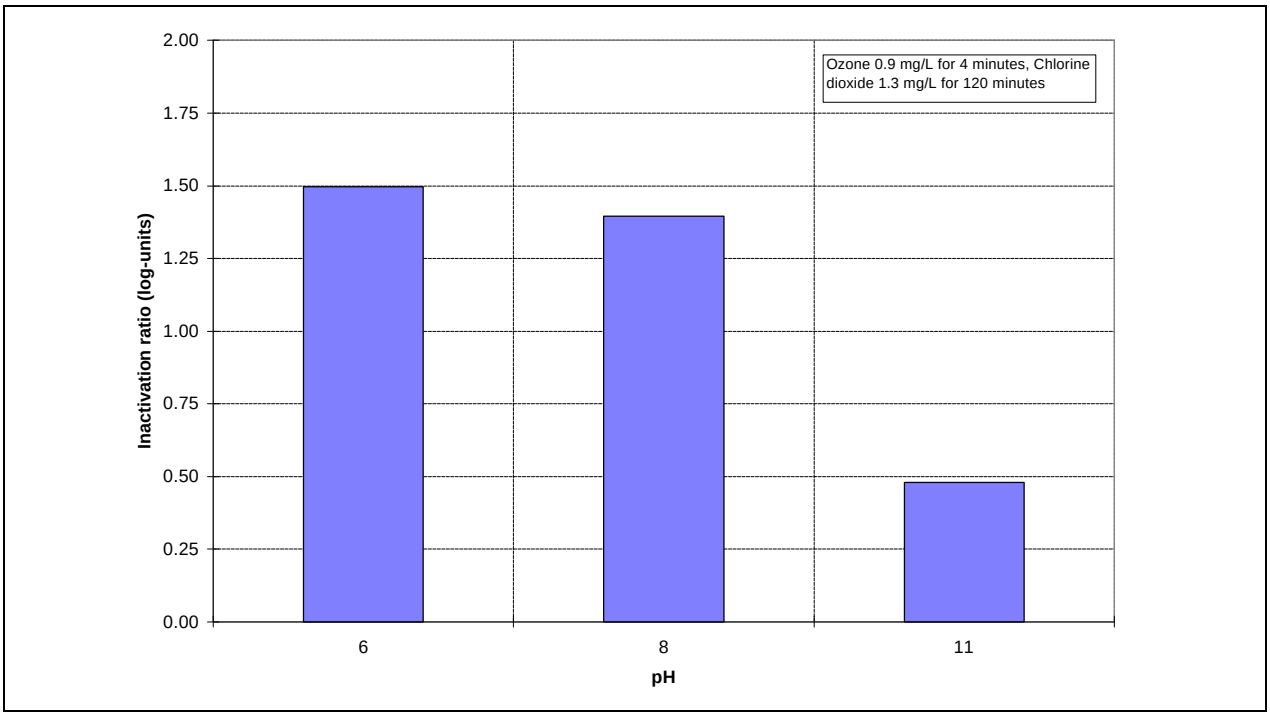
Similar to most chemical disinfectants, the preliminary results from an AWWARF ongoing study suggest that pH and temperature affect the amount of synergistic inactivation achieved by sequential applications of disinfectants (Finch, 1997). The following sections briefly describe the effects these parameters have on pathogen inactivation.

### *9.2.2.1 pH*

The level of inactivation due to the sequential application of chemical disinfectants is believed to be pH dependent (Finch, 1997). Figure 9-1, Figure 9-2, and Figure 9-3 illustrate the impact of pH on the log inactivation of *Cryptosporidium parvum* attributed to synergistic effects for three sequential combinations of ozone-chlorine dioxide, chlorine dioxide-free chlorine, and chlorine dioxide-chloramine, respectively. As shown in these figures, the amount of log inactivation due to synergistic effects is lower at high pH (e.g., pH = 11). These results show that neutral pH is more effective than low pH except for ozone-chlorine dioxide.

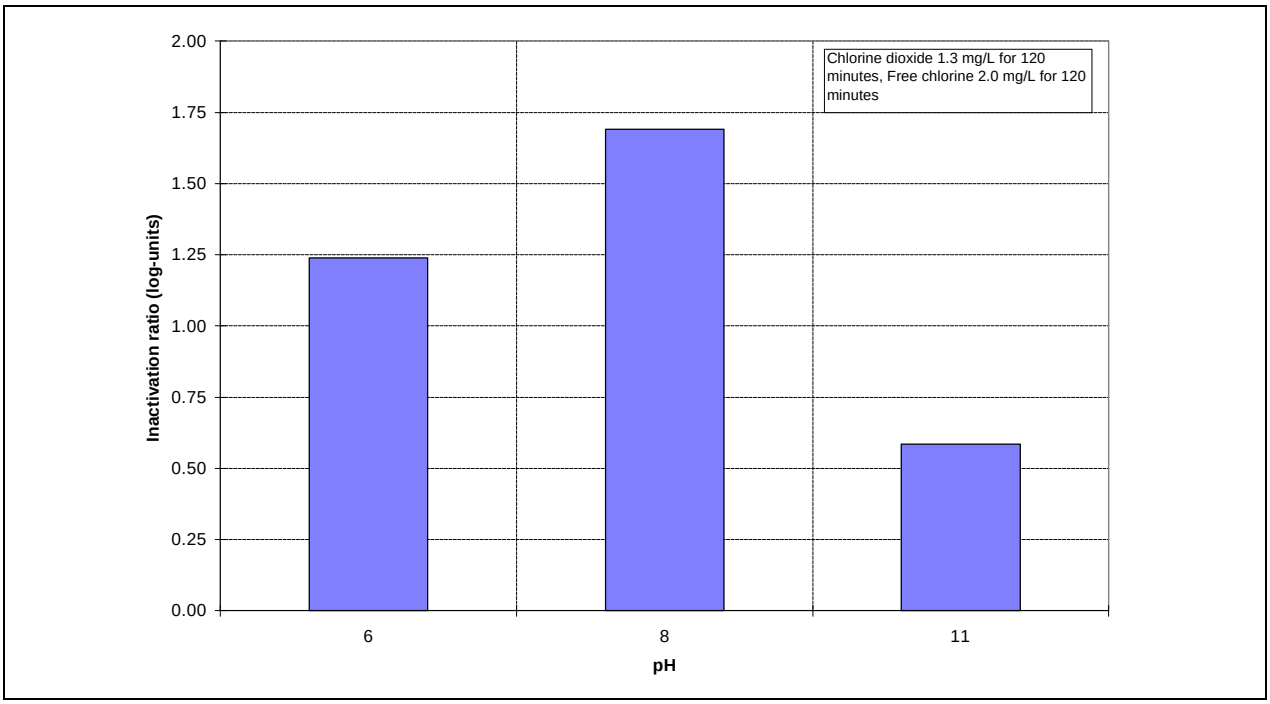
### *9.2.2.2 Combination of Low Temperature and pH*

The combined effect of low temperature and high pH is believed to significantly reduce the amount of *Cryptosporidium* inactivation attributed to synergism (Finch, 1997). One possible explanation for this reduction is that the oocysts contract under these conditions and become harder to penetrate. However, significant reduction in *Cryptosporidium* oocysts inactivation is true under reduced water temperature and high pH whether interactive disinfection is practiced or not. Therefore, reduced inactivation may not be necessarily due to synergism between combined disinfectants.



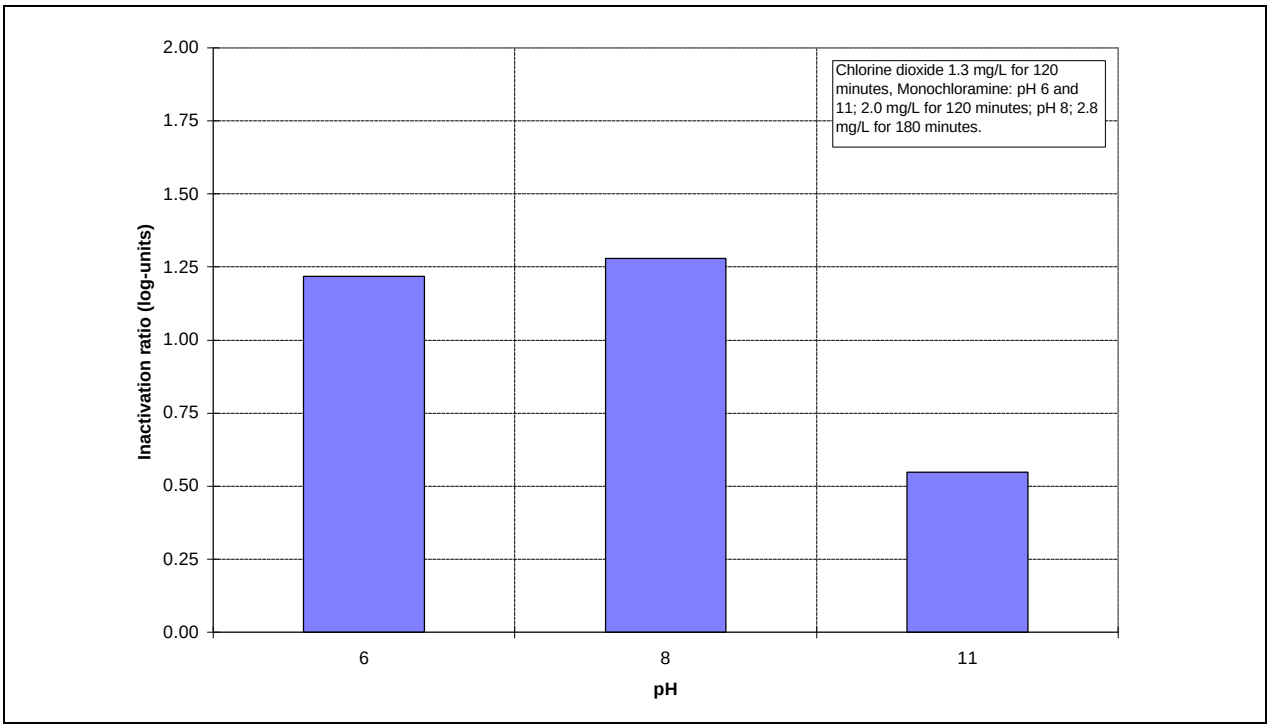

Source: Finch, 1997.

**Figure 9-1. Inactivation of *C. parvum* Attributed to Synergistic Effects. Application of Ozone Followed by Chlorine Dioxide**




Source: Finch, 1997.

**Figure 9-2. Inactivation of *C. parvum* Attributed to Synergistic Effects. Application of Chlorine Dioxide Followed by Free Chlorine**




Source: Finch, 1997.

**Figure 9-3. Inactivation of *C. parvum* Attributed to Synergistic Effects. Application of Chlorine Dioxide Followed by Monochloramine**

### *9.2.2.3 Pathogen Susceptibility*

*Cryptosporidium* oocysts are more susceptible to inactivation by combinations of disinfectants than by individual disinfectants. *Giardia* cysts were also found to have a similar response to:

- Ozone followed by free chlorine;
- Ozone followed by monochloramine;
- Chlorine dioxide followed by free chlorine;
- Chlorine dioxide followed by monochloramine; and
- Free chlorine followed by monochloramine.

However, no synergism was observed with bacterial spores, specifically *Bacillus cereus* spores (Finch, 1997). These results suggest that encysted parasites might show more susceptibility to synergistic effects than bacterial spores. Masking effects caused by turbidity for interactive disinfectants are expected to be similar to those of the individual disinfectants.

## 9.2.3 Pathogen Inactivation Efficiency Using Interactive Disinfectants

Within the last few years there have been several studies to investigate interactive disinfectants. These studies were conducted under various conditions of pH, bench or laboratory scale, and using various organisms:

- Battigelli and Sobsey (1993) studied viral inactivation under lime softening conditions using sequential addition of chlorine and monochloramine;
- Kouame and Haas (1991) determined *E. coli* inactivation during simultaneous addition of free chlorine and monochloramine;
- Finch (1997) is studying various combinations of chlorine, chlorine dioxide, ozone, and monochloramine on inactivation of *Cryptosporidium parvum* oocysts, *Giardia muris* cysts, and *Bacillus cereus* spores under laboratory conditions; and
- Oppenheimer (1997) is developing CT requirements for *Cryptosporidium parvum* inactivation in a variety of natural waters using ozone followed by chlorine.

The following is a summary of the findings of these studies to date.

### *9.2.3.1 Virus Inactivation Using Chlorine and Monochloramine Under High pH*

One of the primary objectives of the Battigelli and Sobsey study (1993) was to evaluate the disinfection efficiency under high pH conditions encountered in conventional lime softening treatment with and without the addition of chlorine and monochloramine. The three microorganisms selected for evaluation were hepatitis A virus, poliovirus 1, and MS-2 coliphage.

During the study, the inactivation kinetics of the three test viruses were determined when 2.0 mg/L monochloramine were formed dynamically after the viruses had been exposed to lime solution and free chlorine for 60 seconds. The authors believed that this approach simulates the conditions typically encountered in a water softening facility where ammonia is applied post-chlorination.

Results indicated that a high degree of inactivation occurs during the first 60 seconds of chlorine addition at approximately 2.4 mg/L free chlorine.

Table 9-6 shows the amount of inactivation attributed to the lime solution, free chlorine, and monochloramine for the three viruses. The table also contains the amount of inactivation attributed to the sequential application of lime solution, free chlorine and monochloramine previously described (simultaneous chloramination). Results shown in Table 9-6 are based on a pH of 11.0 and a total contact time of 360 minutes.

Except for poliovirus 1, the summation of the individual disinfectants was greater than the level of inactivation achieved from the simultaneous chloramination. These results imply that the sequential addition of free chlorine and monochloramine after lime addition to raise the pH to 11, form an

antagonistic (negative) interaction for inactivation of hepatitis A virus and MS-2 coliphage. For poliovirus 1, under similar conditions, an enhancement of 1.4 log inactivation was achieved suggesting positive synergism for poliovirus 1 inactivation.

**Table 9-6. Virus Inactivation By Individual Disinfectants and Simultaneous Chloramination**

| Disinfectant(s) | Log Survival[1] | | | Condition |
|---|---|---|---|---|
| | Hepatitis A Virus | Poliovirus | MS-2 Coliphage | |
| Lime solution only | -3.0 | -0.5 | -4.0 | 360 min contact time |
| Free chlorine | -1.8 | -1.2 | -1.6 | 60 second contact time, 2.5 mg/L chlorine |
| Monochloramine | -3.7 | -1.9 | -3.8 | 2.0 mg/L monochloramine |
| Summation free+monochloramine | -5.5 | -3.1 | -5.4 | Additive |
| Simultaneous chloramination | -4.5 | -4.5 | -4.5 | 2.4 mg/L chlorine 60 second contact time, 2.0 mg/L monochloramine, 359 minutes. |

[1] All data at pH 11 after lime addition
Source: Battigelli and Sobsey, 1993.

### *9.2.3.2 Inactivation of E. coli Under Simultaneous Free and Combined Chlorination*

The inactivation of *E. coli* bacteria by the simultaneous application of free chlorine and monochloramine was investigated at the Illinois Institute of Technology (Kouame and Haas, 1991). Figure 9-4 shows the level of *E. coli* inactivation by free chlorine and monochloramine, separately and combined. The level of inactivation by monochloramine alone after a contact time of 300 seconds was found to be significantly less than that of free chlorine. Therefore, the sum of the individual inactivation by free chlorine and monochloramine was assumed to be equal to that of free chlorine alone. Note that in this case, the residual disinfectant rapidly disappeared due to the breakpoint reactions that occur when monochloramine and free chlorine are combined.

The surviving fraction of bacteria following the simultaneous application of free chlorine and monochloramine is substantially less than what would be expected by adding the individual levels of inactivation. In other words, at similar doses and contact times, the amount of inactivation from the combined disinfectants was greater than the sum of the inactivation due to free chlorine alone and monochloramine alone.

In summary the Kouame and Haas (1991) study showed that high levels of bacteria inactivation can be achieved when free chlorine and monochloramine exist simultaneously in a continuous flow system and that the combined action of both chemicals on the bacteria is synergistic.

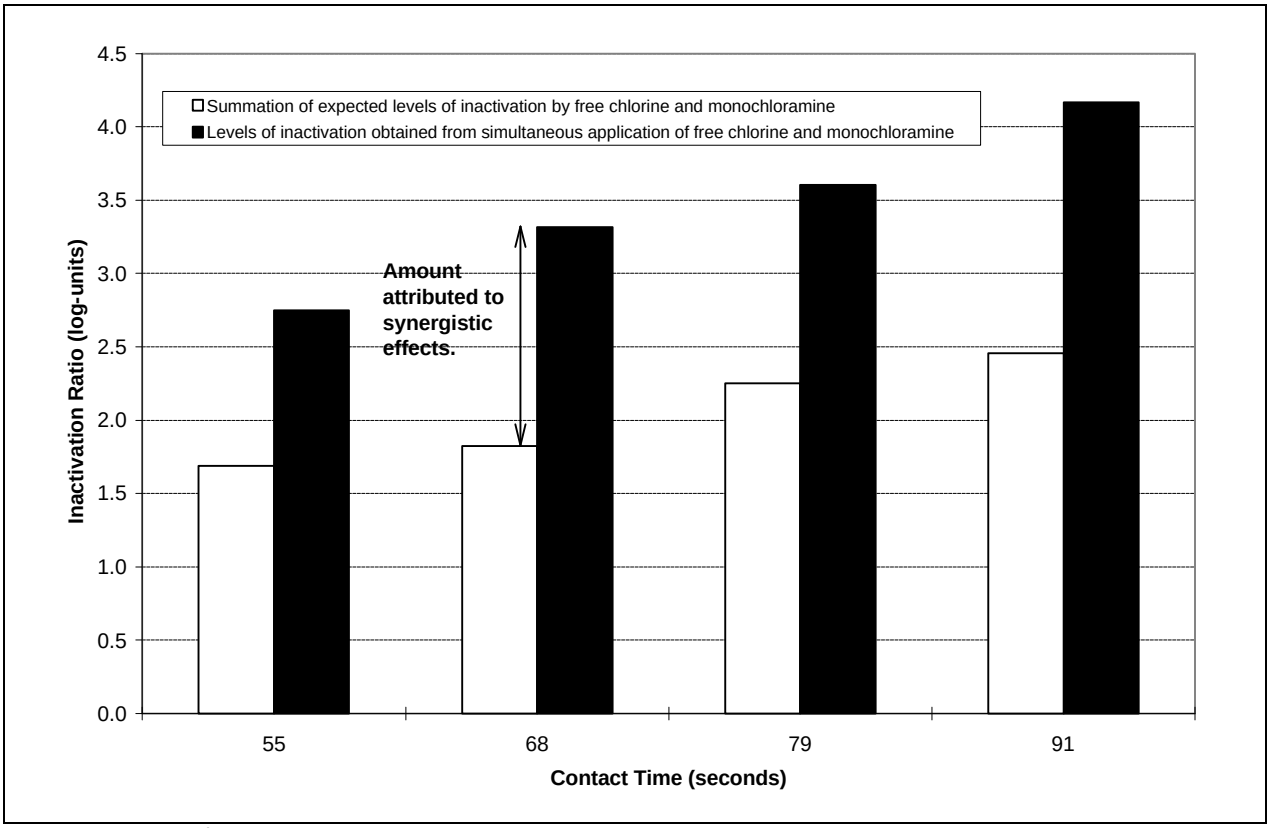
### 9.2.3.3 Cryptosporidium, Giardia, and Bacillus Inactivation in Laboratory Grade Water

The preliminary results of an AWWARF study that investigated the application of multiple disinfectants was presented at a American Water Works Association Technology Transfer Conference in Portland, OR, in August 1997. The objectives of this study were to screen interactive chemical disinfectants (ozone, chlorine, chlorine dioxide, and monochloramine) for inactivation of *Cryptosporidium parvum*, *Giardia muris*, and *Bacillus cereus* and develop design criteria for *Cryptosporidium parvum* inactivation using the best combinations.

#### Ozone Followed By Chlorine Dioxide

Table 9-7 shows the results obtained from ozone and chlorine dioxide application for the inactivation of *Cryptosporidium parvum*.

Based on the data shown in Table 9-7, ozone followed by chlorine dioxide was the most effective disinfectant combination for *Cryptosporidium* inactivation. A total contact time of 124 minutes was required to achieve 3 to 4-log inactivation with ozone and chlorine dioxide residuals of 0.9 and 1.3 mg/L, respectively.




Source: Kouame and Haas, 1991.

**Figure 9-4. Inactivation of *E. coli* Using Free Chlorine and Monochloramine**

## Table 9-7. *C. parvum* Inactivation Using Ozone Followed by Chlorine Dioxide

| Disinfectant | Level of Inactivation (log-units) | | |
|---|---|---|---|
| | pH 6.0 | pH 8.0 | pH 11.0 |
| Ozone | 1.6 | 0.8 | 0 |
| Chlorine dioxide | 0.9 | 1.4 | 2.4 |
| Ozone followed by chlorine dioxide | 4.0 | 3.6 | 2.9 |
| Inactivation attributed to synergism | 1.5 | 1.4 | 0.5 |

Source: Finch, 1997.
Ozone: 0.9 mg/L for 4 minutes, chlorine dioxide 1.3 mg/L for 120 minutes.

### Chlorine Dioxide Followed by Free Chlorine

Table 9-8 through Table 9-10 summarize of the results obtained for chlorine dioxide followed by free chlorine for *Cryptosporidium parvum*, *Giardia muris*, and *Bacillus cereus*, respectively.

## Table 9-8. *C. parvum* Inactivation Using Chlorine Dioxide Followed by Free Chlorine

| Disinfectant | Level of Inactivation (log-units) | | |
|---|---|---|---|
| | pH 6.0 | pH 8.0 | pH 11.0 |
| Chlorine dioxide | 1.0 | 1.4 | 1.6 |
| Free chlorine | 0 | 0 | 0 |
| Chlorine dioxide followed by free chlorine | 2.2 | 3.0 | 2.3 |
| Inactivation attributed to synergism | 1.2 | 1.6 | 0.7 |

Source: Finch, 1997.
Chlorine dioxide 1.3 mg/L for 120 minutes, free chlorine 2.0 mg/L for 120 minutes.

## Table 9-9. *G. muris* Inactivation Using Chlorine Dioxide Followed by Free Chlorine

| Disinfectant | Level of Inactivation (log-units) | |
|---|---|---|
| | pH 6.0 | pH 8.0 |
| Chlorine dioxide | 0.8 | 0.8 |
| Free chlorine | 0.8 | 0.6 |
| Chlorine dioxide followed by free chlorine | 2.2 | 2.0 |
| Inactivation attributed to synergism | 0.6 | 0.6 |

Source: Finch, 1997.
Chlorine dioxide: 1.0 mg/L for 10 minutes, free chlorine 2.0 mg/L for 30 minutes.

## Table 9-10. *B. cereus* Inactivation Using Chlorine Dioxide Followed by Free Chlorine

| Disinfectant | Level of Inactivation (log-units) |
|---|---|
| Chlorine dioxide | 1.8 |
| Free chlorine | 1.2 |
| Chlorine dioxide followed by free chlorine | 2.9 |
| Inactivation attributed to synergism | 0 |

Source: Finch, 1997.
Chlorine dioxide: 2.3 mg/L for 20 minutes, free chlorine for 30 minutes.

Chlorine dioxide followed by free chlorine was capable of achieving between 2 and 3 logs of *Cryptosporidium* inactivation following a total contact time of 240 minutes and approximately 2 logs of inactivation of *Giardia* following only 40 minutes of contact time. No synergism was observed with *Bacillus cereus*. However, approximately 3 logs of inactivation after a contact time of 50 minutes were achieved by the additive effects of chlorine dioxide and free chlorine.

### Chlorine Dioxide Followed by Chloramine

Table 9-11 and Table 9-12 show the results of the inactivation of *Cryptosporidium parvum* and *Giardia muris* when using chlorine dioxide followed by monochloramine.

**Table 9-11. *C. parvum* Inactivation Using Chlorine Dioxide Followed by Chloramine**

| Disinfectant | Level of Inactivation (log-units) | | |
|---|---|---|---|
| | pH 6.0 | pH 8.0 | pH 11.0 |
| Chlorine dioxide | 1.0 | 1.5 | 1.6 |
| Monochloramine | 0 | 0 | 0 |
| Chlorine dioxide followed by monochloramine | 2.2 | 2.8 | 2.1 |
| Inactivation attributed to synergism | 1.2 | 1.3 | 0.5 |

Source: Finch, 1997.
Chlorine dioxide: pH 6, 8, and 11: 1.3 mg/L for 120 minutes. Monochloramine: pH 6 and 11: 2.0 mg/L for 120 minutes, pH 8: 2.8 mg/L for 180 minutes.

**Table 9-12. *G. muris* Inactivation Using Chlorine Dioxide Followed by Chloramine**

| Disinfectant | Level of Inactivation (log-units) | |
|---|---|---|
| | pH 8.0 | pH 11.0 |
| Chlorine dioxide | 0.8 | 0.8 |
| Monochloramine | 0.5 | 0.7 |
| Chlorine dioxide followed by monochloramine | 1.7 | 1.5 |
| Inactivation attributed to synergism | 0.4 | 0 |

Source: Finch, 1997.
pH 8.0: Chlorine dioxide 1.0 mg/L for 5 minutes, monochloramine 2.0 mg/L for 150 minutes.
pH 11.0: Chlorine dioxide 1.0 mg/L for 5 minutes, monochloramine 2.0 mg/L for 5 minutes.

At similar disinfect residuals and contact times, chlorine dioxide followed by monochloramine was found to achieve the same levels of *Cryptosporidium* inactivation as chlorine dioxide followed by free chlorine at pH values of 6 and 11. However, at pH 8, a higher monochloramine residual and contact time were required to achieve inactivation levels comparable to chlorine dioxide and free chlorine. No synergism was found for *Giardia* inactivation at pH 11.0 with only a minimal increase in effectiveness at pH 8.0.

### Ozone Followed by Free Chlorine

Table 9-13 and Table 9-14 show the levels of inactivation of *Giardia muris* and *Bacillus cereus* obtained by using ozone followed by free chlorine.

**Table 9-13. *G. muris* Inactivation Using Ozone Followed by Free Chlorine**

| Disinfectant | Level of Inactivation (log-units) | | |
|---|---|---|---|
| | pH 6.0 | pH 8.0 | pH 11.0 |
| Ozone | 0.5 | 0.8 | 0.4 |
| Free chlorine | 0.8 | 0.6 | 0 |
| Ozone followed by free chlorine | 2.3 | 2.2 | 0.4 |
| Inactivation attributed to synergism | 1.0 | 0.8 | 0 |

Source: Finch, 1997.
pH 6.0: ozone 0.1 mg/L for 60 seconds; free chlorine 2.0 mg/L for 30 minutes.
pH 8.0: ozone 0.1 mg/L for 17 seconds; free chlorine 2.0 mg/L for 60 minutes.
pH 11.0: ozone 0.1 mg/L for 5 seconds; free chlorine 2.0 mg/L for 60 minutes.

**Table 9-14. *B. cereus* Inactivation Using Chlorine Dioxide Followed by Free Chlorine**

| Disinfectant | Level of Inactivation (log-units) |
|---|---|
| Chlorine dioxide | 1.4 |
| Free chlorine | 2.0 |
| Chlorine dioxide followed by free chlorine | 3.4 |
| Inactivation attributed to synergism | 0 |

Source: Finch, 1997.

Ozone followed by free chlorine was capable of achieving approximately 2 logs of *Cryptosporidium* inactivation at pH 6.0 and 8.0; however, only a 0.4 log inactivation was achieved at pH 11.0. The difference in inactivations was primarily caused by the inability of free chlorine to inactivate *Cryptosporidium* at pH 11.0. Similar to the disinfectant combination of chlorine dioxide and free chlorine, no synergism was observed for *Bacillus cereus* inactivation; however, the additive effects of ozone and free chlorine achieved greater than 3 logs of inactivation.

### Ozone Followed by Monochloramine

Table 9-15 shows the results obtained for *Giardia muris* inactivation by ozone followed by monochloramine.

**Table 9-15. *G. muris* Inactivation Using Ozone Followed by Chloramine**

| Disinfectant | Level of Inactivation (log-units) | |
|---|---|---|
| | pH 8.0 | pH 11.0 |
| Ozone | 0.8 | 0.4 |
| Monochloramine | 0.5 | 0.7 |
| Ozone followed by monochloramine | 2.1 | 1.8 |
| Inactivation attributed to synergism | 0.8 | 0.7 |

Source: Finch, 1997.
pH 8.0: ozone 0.1 mg/L for 17 seconds; monochloramine 2.0 mg/L for 150 minutes.
pH 11.0: ozone 0.1 mg/L for 5 seconds; monochloramine 2.0 mg/L for 5 minutes.

Because of the different residuals and contact times, inactivation efficiencies of ozone followed by chloramine and ozone followed by free chlorine could not be compared. However, for similar

monochloramine residuals, a very short monochloramine contact time of 5 minutes at pH 11 was found to achieve a greater inactivation than a contact time of 150 minutes at pH 8.

### Free Chlorine Followed by Monochloramine

Table 9-16 shows the results obtained for *Giardia muris* inactivation by free chlorine followed by monochloramine.

Similar to the results obtained for ozone followed by monochloramine, a very short monochloramine contact time of 5 minutes at pH 11 was found to achieve a greater inactivation than a contact time of 150 minutes at pH 8. However, free chlorine did not achieve any inactivation at pH 11.

**Table 9-16. *G. muris* Inactivation by Free Chlorine Followed by Monochloramine**

| Disinfectant | Level of Inactivation (log-units) | |
|---|---|---|
| | pH 8.0 | pH 11.0 |
| Free chlorine | 0.6 | 0 |
| Monochloramine | 0.5 | 0.7 |
| Free chlorine followed by monochloramine | 2.4 | 0.7 |
| Inactivation attributed to synergism | 1.3 | 0 |

Source : Finch, 1997.
pH 8.0: free chlorine 2.0 mg/L for 60 minutes; monochloramine 2.0 mg/L for 150 minutes.
pH 11.0: free chlorine 2.0 mg/L for 60 minutes, monochloramine 2.0 mg/L for 5 minutes.

### *9.2.3.4 Bench-Scale Tests Using Natural Waters*

In another AWWARF study, Oppenheimer (1997) is developing CT requirements for *Cryptosporidium parvum* inactivation in a variety of natural waters, developing design criteria for full-scale contacting systems from bench scale CT values, and investigating the impact of selected variables on CT requirements. To date, samples have been collected and analyzed from 13 geographically disperse locations. Although a significant amount of data were not available, results from the California State Water Project and Ohio River appear to show that the sequential application of ozone and chloramines resulted in an enhanced inactivation of *C. parvum* as shown in Table 9-17. The sequential application of free chlorine and monochloramine appears to enhance *C. parvum* inactivation by providing some synergistic effects. To obtain the log reduction, however, very high ozone residuals were required which appear to be impractical. In addition, bromate formation was also a problem.

**Table 9-17. *C. parvum* Inactivation by Sequential Application of Ozone and Chloramine**

| Water Source | Ozone | | Chlorine | | Chloramine | | |
|---|---|---|---|---|---|---|---|
| | Residual mg/L | Contact min | Residual mg/L | Contact min | Residual mg/L | Contact min | Log Inactivation Enhancement |
| California State Water Project | 0.8 | 12 | 1.5 | ~0 | 2.5 | 30 | 0.3 to > 1.4 |
| Ohio River | 4 | 15 | 1.5 | 120 | 0.5 | 120 | 0.9 to 1.4 |

Source: Oppenheimer, 1997.

### 9.2.4 Summary: Pathogen Inactivation with Interactive Disinfectants

Various studies have shown the synergistic effects of interactive disinfectants: either simultaneous application or sequential application. The improved disinfection efficiency due to interactive disinfection is variable, ranging from negative (antagonistic) effects (in two studies) to positive enhancement of disinfection efficiency. Many of the studies show definite improvement in inactivation for interactive disinfectants.

Several research projects on the effects of combined disinfectants are underway at the time this manual is being prepared. These projects should provide insight on the mechanisms and applicability of multiple disinfectants. Based on current information, EPA believes that under appropriate situations a positive improvement in disinfection efficiency exists. This enhanced inactivation varies from organism to organism, and with different disinfectant combinations. For the key organisms of interest under normal pH conditions:

- Coliform bacteria inactivation appears to increase with combined disinfectants;
- *Giardia* cyst inactivation appears to increase with combined disinfectants;
- Hepatitis A virus and $MS^{-2}$ coliphage inactivation using combined disinfectants appears to be less efficient than the individual disinfectants;
- Poliovirus 1 inactivation appears to increase with combined disinfectants;
- *Cryptosporidium* oocyst inactivation appears to increase with combined disinfectants; and
- Inactivation of spores appears neutral.

Interactive disinfection is still however considered an emerging technology. As such, CT credits for interactive disinfectants have not yet been established.

## 9.3 Analytical Methods

In general, most of the analytical methods for residual disinfectants are impacted negatively by the presence of other disinfectants. Fortunately, for most of the disinfectants and oxidants listed below, at least one method exists that can be used successfully in the presence of other oxidizing agents. For analytical method details, see the individual disinfectant chapters.

### 9.3.1 Ozone

Residual ozone analysis cannot be performed in the presence of other oxidizing agents including chlorine, chloramine, and potassium permanganate. Typically, the ozone analytical methods exhibit interferences from chlorine, bromine, iodine, and manganese ions. The ACVK is the least susceptible to interference and can be used when manganese concentrations are less than 1 mg/L and free or combined chlorine concentrations are less than 10 mg/L (Gordon et al., 1992).

### 9.3.2 Chlorine Dioxide

Some of the analytical methods for chlorine dioxide, chlorate, and chlorite cannot be performed in the presence of oxidizing agents. Amperometric and iodometric methods cannot be used in the presence of metal ions such as manganese. Analytical methods that can be used in the presence of other disinfectants and oxidants include UV spectrophotometric methods and ion chromatography (Gordon et al., 1992).

### 9.3.3 Potassium Permanganate

The atomic adsorption method for permanganate analysis can be performed in the presence of any of the other disinfectants (Standard Methods, 1995).

### 9.3.4 Chloramine

None of the colorimetric analytical methods for chloramine can be performed in the presence of oxidizing agents such as ozone or hydrogen peroxide. Analytical methods that can be used in the presence of other disinfectants and oxidants include the UV spectrophotometric method and the amperometric titration methods (Gordon et al., 1992).

### 9.3.5 Hydrogen Peroxide

The analytical procedures for hydrogen peroxide in drinking water are all impacted by other oxidizing species such as ozone and chlorine (Gordon et al., 1992).

### 9.3.6 UV Radiation

There are no known interferences from other disinfectants with the measurement of UV radiation (DeMers and Renner, 1992).

### 9.3.7 Summary of Analytical Methods

Ozone analysis in the presence of chlorine is limited. However, these disinfectants are not commonly present simultaneously, especially with the rapid decomposition of ozone.

Hydrogen peroxide analysis is difficult in the presence of ozone and other oxidizing agents. However, when using peroxone, the ozone residual is the analyte used to meet disinfection requirements.

All of the other disinfectants and oxidizing agents can be selectively monitored in the presence of other disinfectants.

## 9.4 Summary

Table 9-18 summarizes, in general, the factors and uses of combined disinfectants. Specific considerations depend on the actual combination of disinfectants used.

**Table 9-18. Summary of Combined Disinfectants**

| Consideration | Description |
|---|---|
| Generation | Generation depends on the type of chemicals used. Ozone, chlorine dioxide, and chloramines require on-site generation. |
| Primary uses | Two separate disinfectants can be used to provide primary and secondary disinfection. By separating the primary and secondary disinfection functions, the processes can be optimized for maximum inactivation and minimum DBP formation.<br><br>Interactive disinfection (using synergism between two disinfectants to enhance inactivation) can serve as a primary disinfectant. |
| Inactivation efficiency | The use of interactive disinfection as primary disinfectant for inactivation of *Giardia, Cryptosporidium,* and viruses are feasible. Interactive disinfection is typically more effective than the individual disinfectants. |
| Byproduct formation | DBP formation is in general reduced by using combined disinfectants. Specifically, continued use of chlorine in combination with other disinfectants can reduce DBP formation. |
| Limitations | Data on the inactivation efficiency of combined disinfectants are still being generated with much information coming from controlled laboratory studies. Additional information is still needed, specifically on full-scale implementation. Dual (primary/secondary) disinfection for DBP control is well established as a preferred treatment option. |
| Point of application | Applied for primary and secondary disinfection. Ozonation should occur after settling and prior to biofiltration. |
| Special considerations | The efficiency and application of combined disinfectants follow to a large extent the limitations and features of the individual disinfectant. The combined disinfectant is often a more effective disinfectant. |

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# APPENDIX A - SUMMARY OF DISINFECTANT USAGE IN THE UNITED STATES

Two sources of information were consulted regarding disinfectant usage in the United States:

- The Community Water Systems Survey (USEPA, 1997); and
- The Information Collection Rule (ICR) database on water utilities (presently under development).

## A.1 Community Water Systems Survey

Most water treatment plants disinfect water prior to distribution. The 1995 Community Water Systems Survey (USEPA 1997a) reports that 81 percent of all community water systems provide some form of treatment on all or a portion of their water sources (Table A-1). The survey found that 99 percent of the surface water systems provide some treatment of their water. Of those systems reporting no treatment, 80 percent rely on ground water as their only water source.

**Table A-1. Disinfection Practices of Water Systems with Treatment**

| Treatment | Service Population | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | <100 | 101-500 | 501-1,000 | 1,001-3,300 | 3,301-10,000 | 10,001-50,000 | 50,001-100,000 | Over 100,001 | Total |
| | | | | Surface Water Systems | | | | | |
| **Pre-Disinfection, Oxidation/Softening** | | | | | | | | | |
| Chlorine | 59.0% | 73.9% | 67.3% | 66.3% | 68.8% | 58.6% | 47.5% | 57.1% | 63.8% |
| Chlorine Dioxide | 0.0% | 0.0% | 0.0% | 5.0% | 4.7% | 13.2% | 14.2% | 7.8% | 6.3% |
| Chloramines | 4.6% | 0.0% | 1.1% | 2.1% | 0.0% | 2.2% | 15.5% | 10.8% | 3.1% |
| Ozone | 0.0% | 0.0% | 0.0% | 0.0% | 0.3% | 0.0% | 5.4% | 5.8% | 0.9% |
| KMnO4 | 0.0% | 4.9% | 9.6% | 9.9% | 15.2% | 28.3% | 25.9% | 28.5% | 16.0% |
| Predisinfection/oxidation | 0.0% | 0.0% | 2.0% | 2.9% | 0.6% | 9.2% | 5.1% | 4.3% | 3.5% |
| Lime/Soda ash softening | 6.8% | 9.8% | 20.9% | 16.2% | 14.3% | 11.7% | 3.5% | 5.9% | 12.5% |
| Recarbonation | 0.0% | 0.0% | 0.0% | 0.0% | 2.1% | 4.7% | 0.6 % | 6.3% | 1.9% |
| **Post-Disinfection** | | | | | | | | | |
| Chlorine | 49.7% | 51.6% | 80.6% | 62.8% | 77.9% | 71.1% | 73.8% | 63.6% | 67.5% |
| Chlorine Dioxide | 0.0% | 0.0% | 0.0% | 0.0% | 0.3% | 4.9% | 5.9% | 11.2% | 1.6% |
| Chloramines | 0.0% | 0.0% | 0.0% | 2.9% | 2.1% | 15.6% | 29.4% | 24.2% | 8.1% |
| Postdisinfection combine | 0.0% | 0.0% | 0.0% | 2.1% | 4.0% | 3.9% | 1.9% | 1.6% | 3.0% |
| Fluoridation | 0.0% | 4.9% | 13.9% | 32.4% | 42.6% | 48.8% | 49.9% | 63.6% | 35.5% |
| | | | | Ground Water Systems | | | | | |
| **Pre-Disinfection, Oxidation/Softening** | | | | | | | | | |
| Chlorine | 64.2% | 69.9% | 56.7% | 73.2% | 60.6% | 57.4% | 36.2% | 38.1% | 63.9% |
| Chlorine Dioxide | 1.3% | 0.0% | 0.0% | 0.0% | 0.0% | 0.0% | 3.1% | 0.0% | 0.3% |
| Chloramines | 0.0% | 0.0% | 0.0% | 0.0% | 0.0% | 0.6% | 1.4% | 0.7% | 0.1% |
| Ozone | 0.0% | 0.0% | 0.0% | 0.0% | 0.0% | 0.0% | 0.0% | 0.6% | 0.0% |
| KMnO4 | 0.0% | 0.9% | 2.2% | 0.6% | 5.8% | 3.2% | 7.0% | 0.0% | 1.8% |
| Predisinfection/oxidation | 0.3% | 0.5% | 0.0% | 0.7% | 1.0% | 2.6% | 0.0% | 0.0% | 0.7% |
| Lime/Soda ash softening | 2.9% | 2.9% | 2.2% | 3.6% | 3.5% | 3.8% | 5.0% | 9.1% | 3.2% |
| Recarbonation | 0.0% | 0.5% | 0.0% | 0.6% | 1.4% | 1.5% | 2.8% | 1.1% | 0.6% |
| **Post-Disinfection** | | | | | | | | | |
| Chlorine | 23.0% | 23.4% | 32.5% | 28.3% | 42.5% | 41.9% | 54.5% | 65.8% | 31.0% |
| Chlorine Dioxide | 0.0% | 1.0% | 0.0% | 0.0% | 0.0% | 0.6% | 0.0% | 0.0% | 0.4% |
| Chloramines | 0.0% | 0.0% | 0.0% | 0.0% | 0.1% | 1.1% | 3.9% | 4.3% | 0.3% |
| Postdisinfection combine | 0.0% | 0.0% | 0.0% | 0.0% | 0.1% | 0.1% | 0.0% | 0.0% | 0.0% |
| Fluoridation | 2.4% | 6.3% | 13.2% | 12.4% | 45.3% | 31.2% | 34.3% | 52.5% | 16.0% |

Source: 1995 Community Water Systems survey (USEPA, 1997a)

## A.2 Information Collection Rule Database

The ICR database contains information from 527 community water systems. Some of these systems are owned by the same utility, representing different treatment plants within the same organization. The following tables summarize the ICR data regarding disinfectant database entries. Note that the information about disinfectant includes usage of the disinfectant at the given utility. In some cases, the utility may use more than one disinfectant. Therefore, the 527 systems reported use of 740 different disinfectants. Three systems reported using 3 different chemical disinfectants - for different purposes or during different times of the year. The tables show a breakdown of the disinfectant usage at these facilities. Capacities are shown in terms of flow. Population served data are not recorded.

The tables are as follows:

- Table A-2 shows the breakdown of systems based on water source (i.e., surface or ground water).
- Table A-3 to Table A-5 shows the disinfectant usage at water plants by flow categories for all water sources, surface water sources, and ground water sources, respectively. The tables show the percentage of facilities that are using the particular disinfectant. Because some facilities use more than one disinfectant, the total usage exceeds 100 percent. For example, for facilities in the range 51-100 mgd, the total disinfectant usage is 155 percent. This means that 155 types of disinfection systems are used at 100 plants.
- Table A-6 through Table A-8 shows the disinfectant usage in number of applications in water plants by flow categories for all water sources, surface water sources, and ground water sources, respectively. These tables show the actual numbers used in calculating the percentages in Table A-3 to Table A-5.
- Table A-9 through Table A-11 show the number of systems using two or more disinfectants for all water sources, surface water sources, and ground water sources, respectively. The database does not separate usage as primary or secondary disinfectant.

**Table A-2. Breakdown of systems in Survey based on Water Source**

| Systems | GW | SW | GW/SW | Unknown | Total |
|---|---|---|---|---|---|
| Number | 135 | 390 | 2 | 2 | 529 |
| Percentage | 26% | 74% | 0% | 0% | 100% |

GW = Ground Water; SW = Surface Water; GW/SW = Choice of Ground or Surface Water Source.

**Table A-3. Disinfectant usage as a function of flow for all Water Sources. Numbers show the percentage of systems using a particular disinfectant**

| Flow, mgd | $Cl_2$ | NaOCl | $NH_2Cl_2$ | $O_3$ | $ClO_2$ | Total use |
|---|---|---|---|---|---|---|
| 0-5 | 69% | 19% | 19% | 0% | 0% | 106% |
| 6-10 | 90% | 10% | 10% | 0% | 0% | 110% |
| 11-50 | 93% | 6% | 30% | 5% | 9% | 143% |
| 51-100 | 95% | 5% | 41% | 5% | 9% | 155% |
| >100 | 98% | 5% | 43% | 5% | 3% | 154% |
| Unknown | 77% | 13% | 8% | 0% | 0% | 98% |
| Percentage* | 92% | 7% | 31% | 4% | 6% | 140% |

Percentage calculated as a fraction of 527 - the total number of systems. 740 different disinfectants are used by the 527 systems.

**Table A-4. Disinfectant usage as a function of flow for Surface Water Sources. Numbers show the percentage of systems using a particular disinfectant**

| Flow, mgd | $Cl_2$ | NaOCl | $NH_2Cl$ | $O_3$ | $ClO_2$ | Total use |
|---|---|---|---|---|---|---|
| 0-5 | 89% | 11% | 0% | 0% | 0% | 100% |
| 6-10 | 85% | 8% | 8% | 0% | 0% | 100% |
| 11-50 | 93% | 5% | 32% | 5% | 12% | 148% |
| 51-100 | 96% | 4% | 42% | 6% | 11% | 158% |
| >100 | 96% | 6% | 46% | 6% | 4% | 158% |
| Unknown | 88% | 13% | 50% | 0% | 0% | 150% |
| Percentage* | 94% | 5% | 37% | 5% | 9% | 150% |

Percentage calculated as a fraction of 527 - the total number of systems. 576 different disinfectants are used by the 383 systems.

**Table A-5. Disinfectant usage as a function of flow for Ground Water Sources. Numbers show the percentage of systems using a particular disinfectant**

| Flow, mgd | $Cl_2$ | NaOCl | $NH_2Cl$ | $O_3$ | $ClO_2$ | Total use |
|---|---|---|---|---|---|---|
| 0-5 | 43% | 29% | 43% | 0% | 0% | 114% |
| 6-10 | 94% | 13% | 13% | 0% | 0% | 119% |
| 11-50 | 92% | 6% | 24% | 2% | 2% | 125% |
| 51-100 | 88% | 13% | 38% | 0% | 0% | 138% |
| >100 | 108% | 0% | 25% | 0% | 0% | 133% |
| Unknown | 79% | 14% | 0% | 0% | 0% | 93% |
| Percentage* | 87% | 10% | 18% | 1% | 1% | 117% |

Percentage calculated as a fraction of 527 - the total number of systems. 168 different disinfectants are used by the 144 systems.

**Table A-6. Disinfectant usage as a function of flow for all Water Sources**

| Flow, mgd | $Cl_2$ | NaOCl | $NH_2Cl$ | $O_3$ | $ClO_2$ | Total use | Total plants |
|---|---|---|---|---|---|---|---|
| 0-5 | 11 | 3 | 3 | 0 | 0 | 17 | 16 |
| 6-10 | 26 | 3 | 3 | 0 | 0 | 32 | 29 |
| 11-50 | 200 | 12 | 65 | 10 | 20 | 307 | 215 |
| 51-100 | 113 | 6 | 49 | 6 | 11 | 185 | 119 |
| >100 | 94 | 5 | 41 | 5 | 3 | 148 | 96 |
| Unknown | 40 | 7 | 4 | 0 | 0 | 51 | 52 |
| Total | 484 | 36 | 165 | 21 | 34 | 740 | 527 |
| Percentage* | 92% | 7% | 31% | 4% | 6% | 140% | |

* Percentage calculated as a fraction of 527 - the total number of systems. 740 different disinfectants are used by the 527 systems.

**Table A-7. Disinfectant usage as a function of flow for Surface Water Sources**

| Flow, mgd | $Cl_2$ | NaOCl | $NH_2Cl$ | $O_3$ | $ClO_2$ | Total use | Total plants |
|---|---|---|---|---|---|---|---|
| 0-5 | 8 | 1 | 0 | 0 | 0 | 9 | 9 |
| 6-10 | 11 | 1 | 1 | 0 | 0 | 13 | 13 |
| 11-50 | 154 | 9 | 53 | 9 | 20 | 245 | 165 |
| 51-100 | 99 | 4 | 43 | 6 | 11 | 163 | 103 |
| >100 | 82 | 5 | 39 | 5 | 3 | 134 | 85 |
| Unknown | 7 | 1 | 4 | 0 | 0 | 12 | 8 |
| Total | 361 | 21 | 140 | 20 | 34 | 576 | 383 |
| Percentage* | 94% | 5% | 37% | 5% | 9% | 150 | |

* Percentage calculated as a fraction of 383 - the total number of plants. 576 different disinfectants are used by the 383 systems.

**Table A-8. Disinfectant usage as a function of flow for Ground Water Sources**

| Flow, mgd | $Cl_2$ | NaOCl | $NH_2Cl$ | $O_3$ | $ClO_2$ | Total use | Total plants |
|---|---|---|---|---|---|---|---|
| 0-5 | 3 | 2 | 3 | 0 | 0 | 8 | 7 |
| 6-10 | 15 | 2 | 2 | 0 | 0 | 19 | 16 |
| 11-50 | 47 | 3 | 12 | 1 | 1 | 64 | 51 |
| 51-100 | 14 | 2 | 6 | 0 | 0 | 22 | 16 |
| >100 | 13 | 0 | 3 | 0 | 0 | 16 | 12 |
| Unknown | 33 | 6 | 0 | 0 | 0 | 39 | 42 |
| Total | 125 | 15 | 26 | 1 | 1 | 168 | 144 |
| Percentage* | 87% | 10% | 18% | 1% | 1% | 117% | |

* Percentage calculated as a fraction of 144 - the total number of systems. 168 different disinfectants are used by the 144 systems.

**Table A-9. Number Water Systems (Ground and Surface Water Sources) using Two Different Disinfectants**

| | $Cl_2$ | NaOCl | $NH_2Cl$ | $O_3$ | $ClO_2$ |
|---|---|---|---|---|---|
| $Cl_2$ | --- | 8 | 150 | 17 | 32 |
| NaOCl | --- | --- | 8 | 3 | 0 |
| NH2Cl | --- | --- | --- | 12 | 18 |
| $O_3$ | --- | --- | --- | --- | 1 |
| $ClO_2$ | --- | --- | --- | --- | --- |

**Table A-10. Number Surface Water Systems using Two Different Disinfectants**

| | $Cl_2$ | NaOCl | $NH_2Cl$ | $O_3$ | $ClO_2$ |
|---|---|---|---|---|---|
| $Cl_2$ | --- | 7 | 129 | 16 | 31 |
| NaOCl | --- | --- | 7 | 7 | 3 |
| NH2Cl | --- | --- | --- | 11 | 18 |
| $O_3$ | --- | --- | --- | --- | 1 |
| $ClO_2$ | --- | --- | --- | --- | --- |

**Table A-11. Number Ground Water Systems using Two Different Disinfectants**

| | $Cl_2$ | NaOCl | $NH_2Cl$ | $O_3$ | $ClO_2$ |
|---|---|---|---|---|---|
| $Cl_2$ | --- | 1 | 21 | 1 | 0 |
| NaOCl | --- | --- | 1 | 0 | 0 |
| $NH_2Cl$ | --- | --- | --- | 1 | 0 |
| $O_3$ | --- | --- | --- | --- | 0 |
| $ClO_2$ | --- | --- | --- | --- | --- |

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# Appendix B – Selected Costs of Alternative Disinfection Systems

## B.1 Technologies and Costs for Control of Disinfection By-Products

Costs were developed for modifying a "base" or "typical" treatment plant to add disinfection and other technologies. The base plant is described as a conventional treatment plant using chlorine/chlorine disinfection consisting of rapid mixing, flocculation, sedimentation, chlorination, filtration, contact basin, chemical feed systems and finished water storage. This appendix contains figures and tables from *Technologies and Costs for Control of Disinfection By-Products* (USEPA, 1998), retaining the report's original figure and table numbers. Incremental costs are shown, determined by calculating the cost for the modified treatment plant and subtracting the base treatment plant cost.

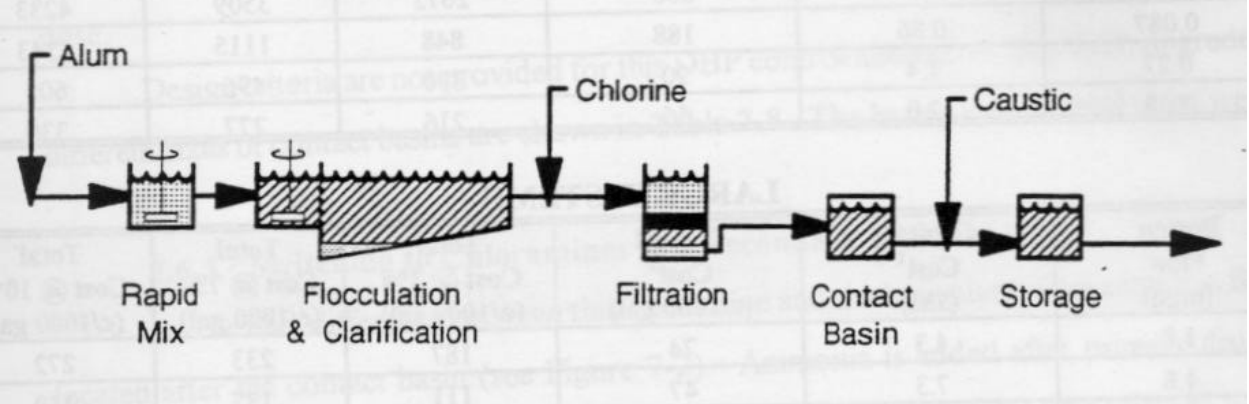
The base treatment plant shown in Figure 7-1, is a basic alum coagulation and filtration plant, with chlorine disinfection. This plant was modified to meet disinfection requirements. The bases for the cost estimates are shown in Tables 7-3, 7-4, 7-5, and 7-6. The 12 flow categories for which the costs were determined are shown in Table 7-2.

Schematics and costs to add the following schemes are shown in the attached figures and tables.

- Base treatment plant - Figure 7-1 and Table 7-7.
- Move point of chlorination. This modification assumes no cost for moving the chlorine addition point, but costs for an added contact basin are shown in Table 7-8.
- Change to Chlorine/Chloramine - Figure 7-2 and Table 7-9.
- Change to Ozone/Chloramine - Figure 7-3 and Table 7-12.
- Change to Chlorine Dioxide – Table 7-13.

See USEPA, 1998 for more details and information upon the costs of other technologies.

## FIGURE 7-1. ALUM COAGULATION / FILTRATION BASE PLANT




## TABLE 7-2. EPA FLOW CATEGORIES

| EPA FLOW CATEGORIES | POPULATION | AVERAGE FLOW mgd) | CAPACITY mgd) |
|---|---|---|---|
| | | | |
| 1 | | 0.0056 | |
| 2 | | 0.024 | |
| 3 | | 0.086 | |
| 4 | | 0.23 | |
| **LARGE SYSTEMS - DESIGN FLOW > 1 MGD** | | | |
| | 5,500 | | 1.8 |
| | 15,000 | | 4.8 |
| | 35,000 | | 11 |
| | 60,000 | | 18 |
| | 88,000 | | 26 |
| | 175,000 | | 51 |
| | 73 | 120 | |
| 12 | | 270 | |
| 12a | | 350 | |

**TABLE 7-3. SMALL SYSTEMS BASIS FOR COST ESTIMATES FOR DBP CONTROL**

| Process | WATER Model Assumptions | Engineering Assumption |
|---|---|---|
| Package Raw Water Pumping | Premanufactured packaged pumping station using submersible pump contained in a 20 ft deep steel pump sump.<br><br>Manifold piping, sump intake valve, pump check valves, and electrical controls.<br><br>Total dynamic head is 50 ft.<br><br>Pump and motor efficiencies are 80 and 90%, respectively. | |
| Package Complete Treatment Plant | Coagulation, flocculation, sedimentation, and filtration equipment provided including tube settlers rated at 1,500 gpd/sf, mixed media filters with application rates of 2 to 5 gpm/sf and media depth of 30 in.<br><br>Chemical feed facilities include storage tanks and feed pumps.<br><br>Filter backwash pumps and, where applicable, surface wash water pumps.<br><br>Flow measurement and control devices, pneumatic air supply (for 200 gpm or larger plants), effluent pumps, and building. | |
| Hypochlorite Solution | Solution tanks, mixers, and metering pumps | Sodium hypochlorite dose of 2.4 mg/L determined by WTP model. |
| Chlorination System | Metering pumps, PVC pipes, valves and controls are included. | |
| Sodium Hydroxide Feed System | Storage tanks, heater, manual transfer pump, mixers, feed tanks and metering pumps are included.<br><br>PVC pipes, valves and control are also included. | |
| Alum Feed System | Solution tanks, mixers, and calibrated metering pumps are included.<br><br>PVC pipes, valves and controls are also included. | Alum dose is determined by DBP control alternatives. |
| | | |

**TABLE 7-3. SMALL SYSTEMS BASIS FOR COST ESTIMATES FOR DBP CONTROL**

| Process | WATER Model Assumptions | Engineering Assumption |
|---|---|---|
| Package High Service Pump Station | Includes 2 or 3 centrifugal pumps, pressure sensing, flow control valves, instrumentation and equipment.<br><br>Pumps provide a maximum output of 70 psi. | |
| Clearwell Storage Above Ground | Above ground, steel tanks including instrumentation and control of clearwell water level and instrumentation for turbidity and residual monitoring is provided. | Clearwell size is based on storage of 25% of the daily operating flow. |
| Sludge Dewatering Lagoons | Unlined lagoon and inlet, outlet structures are provided.<br><br>2 ft freeboard, 3:1 side slopes, 5 ft depth are also provided. | Sizing of lagoons is based on solids content of 5%.<br><br>Sludge is thickened to a solids concentration of 30%. |
| Dewatered Sludge Hauling | Loading facilities including sludge conveyer, hopper, and hopper enclosure are provided.<br><br>Length of haul is 20 miles one-way. | |
| Contact Basin | | Below ground tanks without repumping are assumed.<br><br>Size of basin is 60 minutes, as determined by the WTP model.<br><br>The well baffled tanks are assumed to provide actual contact time of 0.7 times the theoretical according to the SWTR Guidance Manual.<br><br>O&M costs were unpredictable and were assumed to be negligible. |

## TABLE 7-4. LARGE SYSTEMS BASIS FOR COST ESTIMATES FOR DBP CONTROL

| Process | WATER Model Assumptions | Engineering Assumption |
|---|---|---|
| Raw Water Pumping | Total dynamic head 100 ft<br><br>Manifold piping velocity<br><br>Standby pump, manifold piping, and instrumentation are provided | |
| Alum Feed System | Diaphragm metering pumps, steel storage hoppers with dust collector, and mechanical weight belt feeders<br><br>Commercial alum density 60 lb/cu ft<br><br>Dissolving tank detention time 5 min with 2 gal of water per lb of dry alum added<br><br>Maximum hopper volume 6,000 cu ft with fifteen days of storage | Alum dose is determined by DBP control alternatives |
| Rapid Mix | Vertical shaft, variable speed turbine mixers with stainless steel shafts and paddles and TEFC motors<br><br>Maximum basin capacity 2,500 cu ft<br><br>Water temperature 15" C<br>Overall mechanism efficiency 70% | G = 900/sec<br><br>Detention time is 1 min at design flow |
| Flocculation (Horizontal Paddle) | Rectangular-shaped, reinforced concrete basins with 12 ft depth, 4:1 length to width ratio, and 12,500 cu ft individual maximum basin size<br><br>Variable speed drive units requiring 15 min/day routine O&M and an oil change every 6 months requiring 4 hrs of labor<br><br>Overall mechanism efficiency 60% | G = 50/sec<br><br>Detention time is 30 min at design flow |
| Rectangular Clarifiers | Chain and flight collector with drive mechanism, sludge pumps, reinforced concrete structure, and withdraw pumps are included | Overflow rate = 1,000 gpd/sq ft |

## TABLE 7-4. LARGE SYSTEMS BASIS FOR COST ESTIMATES FOR DBP CONTROL

| Process | WATER Model Assumptions | Engineering Assumption |
|---|---|---|
| | Side wall depth = 12 ft | Maximum number of units is 2<br>Maximum basin area = 20,000 sq ft |
| Gravity Filtration Systems | Filter structure, underdrains, wash water troughs, pipe gallery piping and valves, instrumentation, control panel, and filter housing are provided<br><br>Filter box depth = 16 ft<br><br>Maximum filter size = 1,275 sq ft | Minimum 4 filters per plant<br><br>Filter loading rate = 4 gpm/sq ft |
| Filtration Dual Media | 20 in of 1.0 to 1.2 mm effective size anthracite coal (UC = 1.7)<br><br>10 in of 0.42 to 0.52 mm effective size silica sand (UC = 1.6)<br><br>12 in underdrains. Media consisting of 4 sizes of silica gravel | |
| Backwash Pumping Facilities | All required pumps and motors, flow control, sequencing control, valves and backwash headers are included<br><br>Pumping head = 50 ft<br><br>Overall mechanism efficiency 70% | Backwash rate = 18 gpm/sq ft<br><br>One filter is backwashed at a time with each filter backwashed approximately every two days |
| Wash Water Surge Basins | Below ground, reinforced concrete basins and level control instrumentation provided | Sized to store a 20 min volume of backwash water at design flow |
| Unthickened Sludge Pumping | Variable speed, centrifugal pumps, piping and valves, electrical equipment housing, dry well, and a wet well are included<br><br>Pipe velocity = 5 ft/sec<br><br>Total dynamic head = 30 ft | Unthickened sludge solids concentration = 1%<br><br>12 hr/day of sludge pumping |

## TABLE 7-4. LARGE SYSTEMS BASIS FOR COST ESTIMATES FOR DBP CONTROL

| Process | WATER Model Assumptions | Engineering Assumption |
|---|---|---|
| | Overall pump-motor efficiency = 65% | |
| Sludge Dewatering Lagoons | Unlined lagoon and inlet and outlet structures are provided<br><br>2 ft freeboard, 3:1 side slopes, and 10 ft depth are also provided | Solids production is determined by WTP model<br><br>Sludge is thickened to a solids concentration of 30%<br><br>Sizing of lagoons is based on a solids content of 5% |
| Dewatered Sludge Hauling | Loading facilities including sludge conveyor, hopper, and hopper enclosure are provided<br><br>Length of haul is 20 miles one-way | Dewatered sludge has a solids content of 30% |
| In-plant Pumping | Constant speed, vertical turbine pumps, pump motor, wet well, and piping and valves are included<br><br>Pipe velocity = 5 ft/sec | Total dynamic head = 50 ft |
| Chlorine Feed Facilities | Chlorinator, standby chlorinator, cylinder scales, evaporators, residual analyzers with flow proportioning device injector pumps, and housing to include 30 days of cylinder storage are provided<br><br>Injector pumps deliver water at 25 psi to allow production of 3,500 mg/L solution | Chlorine dose is 2.4 mg/L as determined by WTP model |
| Sodium Hydroxide Feed System | Storage tanks, heater, manual transfer pump mixers, feed tanks and metering pumps are included<br><br>PVC pipes, valves and controls are also included | Sodium hydroxide dose is 16 mg/L as determined by WTP model |
| Finished Water Pumping | Vertical turbine pumps powered by constant speed motors, electrical equipment instrumentation, valves, and manifolds are provided<br><br>Total dynamic head is 300 ft | |

**TABLE 7-4. LARGE SYSTEMS BASIS FOR COST ESTIMATES FOR DBP CONTROL**

| Process | WATER Model Assumptions | Engineering Assumption |
|---|---|---|
| | Standby pump is also included | |

## TABLE 7-5. COST ALLOWANCE FACTORS

| Item | Small Systems Water Model (%) | Large Systems WATERCO$T Model(%) |
|---|---|---|
| Site work and Interface Piping | 10 | 15 |
| Subsurface Considerations | 10 | 10 |
| Standby Power | 5 | 5 |
| General Contractors Overhead and Profit | 12 | 12 |
| Engineering | 15(1) | 15(2) |
| Legal, Fiscal and Administration fees | 5 to 6(1) | 9 to 11(2) |

Notes:

(1)Percentages added to estimated construction cost plus estimated cost for other allowances factors.

(2)Percentages added to estimated construction cost only.

## TABLE 7-6. INDICES USED IN THE ESCALATION OF COSTS

| DESCRIPTION | INDEX REFERENCE | NUMERICAL VALUE | ESCALATION VALUE |
|---|---|---|---|
| **Building Cost Index** | ENR [1] | 3391.86 | 1.23 |
| **Chemical & Allied Products** | BLS [2] | 147.2 | 1.19 |
| **Skilled Labor** | ENR [1] | 5231.35 | 1.178 |
| **Materials** | ENR [1] | 2268.57 | 1.328 |
| **Utility Natural Gas** | BLS 055 [2] | 111.3 | 1.679 |

[1] Engineering News Record (July, 1997)

[2] Bureau of Labor Statistics (March, 1997)

## TABLE 7-7. ESTIMATED BASE PLANT COSTS

### SMALL SYSTEMS

| Design Flow (mgd) | Capital Cost[1] ($M) | O & M Cost[2] (¢/1000 gal) | Total Cost @ 3% (¢/1000 gal) | Total Cost @ 7% (¢/1000 gal) | Total Cost @ 10% (¢/1000 gal) |
|---|---|---|---|---|---|
| 0.024 | 0.63 | 600 | 2672 | 3509 | 4233 |
| 0.087 | 0.86 | 188 | 848 | 1115 | 1343 |
| 0.27 | 1.4 | 90 | 390 | 496 | 605 |
| 0.65 | 2.0 | 56 | 216 | 277 | 330 |

### LARGE SYSTEMS

| Design Flow (mgd) | Capital Cost[1] ($M) | O & M Cost[2] (¢/1000 gal) | Total Cost @ 3% (¢/1000 gal) | Total Cost @ 7% (¢/1000 gal) | Total Cost @ 10% (¢/1000 gal) |
|---|---|---|---|---|---|
| 1.8 | 4.3 | 74 | 187 | 233 | 272 |
| 4.8 | 7.3 | 47 | 111 | 137 | 159 |
| 11 | 12 | 39 | 83 | 102 | 118 |
| 18 | 17 | 36 | 72 | 86 | 98 |
| 26 | 22 | 35 | 66 | 78 | 89 |
| 51 | 36 | 33 | 58 | 68 | 76 |
| 210 | 120 | 32 | 50 | 58 | 65 |
| 430 | 230 | 31 | 47 | 53 | 59 |
| 520 | 380 | 26 | 46 | 54 | 61 |

[1] 1991 Cost escalated based upon a factor of 1.23 derived from the ENR BCI

[2] 1991 Cost escalated based upon a factor of 1.19 derived from the BLS Chemical and Allied Products Index

**TABLE 7-8. ESTIMATED UPGRADE COSTS FOR ADDITIONAL CONTACT BASIN SIZE (x $1000)** [1]

| DESIGN FLOW | Chlorine Contact Basin Time | | | | | | |
|---|---|---|---|---|---|---|---|
| | 30 min | 60 min | 120 min | 180 min | 240 min | 300 min | 360 min |
| 0.024 | 14 | 21 | 26 | 28 | 38 | 46 | 55 |
| 0.087 | 25 | 34 | 66 | 76 | 82 | 84 | 100 |
| 0.27 | 52 | 80 | 103 | 140 | 180 | 220 | 234 |
| 0.65 | 77 | 112 | 218 | 251 | 284 | 317 | 351 |
| 1.8 | 197 | 244 | 335 | 427 | 519 | 611 | 702 |
| 4.8 | 274 | 396 | 642 | 887 | 1,132 | 1,376 | 1622 |
| 11 | 432 | 713 | 1,274 | 1,836 | 2,399 | 2,961 | 3521 |
| 18 | 611 | 1,070 | 1,990 | 2,909 | 3,828 | 4,748 | 5667 |
| 26 | 815 | 1,478 | 2,807 | 4,135 | 5,462 | 6,791 | 8118 |
| 51 | 1,454 | 2,755 | 5,360 | 7,965 | 10,569 | 13,175 | 15,778 |
| 210 | 5514 | 10,876 | 21,600 | 32,324 | 43,050 | 53,774 | 64,499 |
| 430 | 11,132 | 22,112 | 44,071 | 66,031 | 87,991 | 109,951 | 131,910 |
| 520 | 13,374 | 26,639 | 53,224 | 79,785 | 106,352 | 132,193 | 159,456 |

[1] 1991 Cost escalated based upon a factor of 1.23 derived from the ENR BCI

**FIGURE 7-2. ALUM COAGULATION / FILTRATION SYSTEM UPGRADED WITH CHLORINE / CHLORAMINE DISINFECTION**

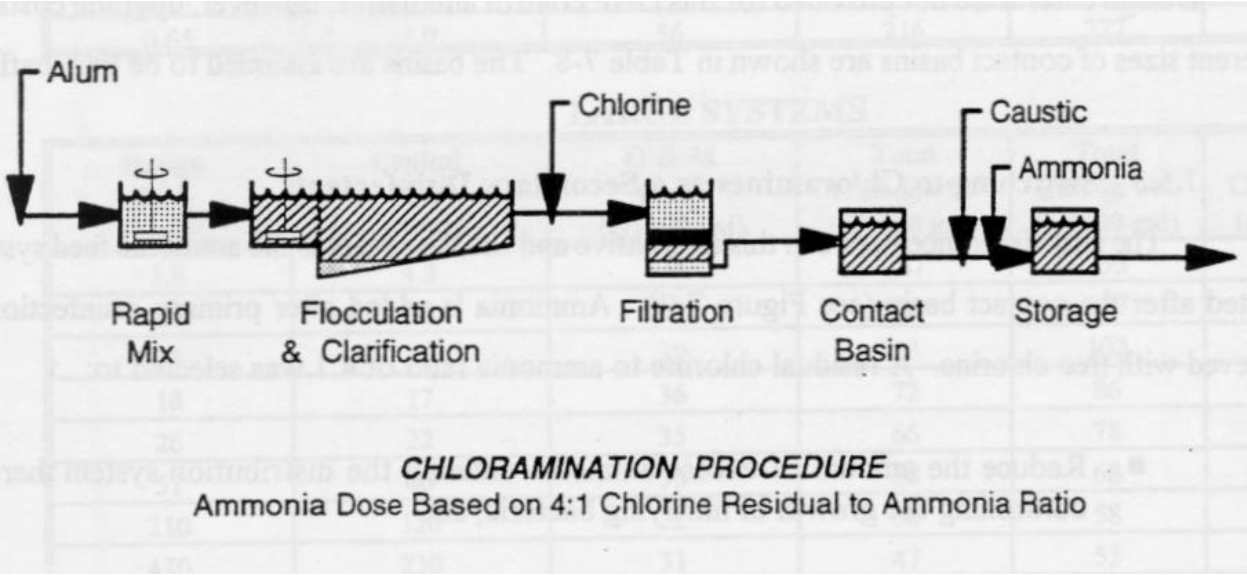



**TABLE 7-9. ESTIMATED UPGRADE COSTS FOR CHLORAMINES AS SECONDARY DISINFECTANT**

**SMALL SYSTEMS**

| Design Flow (mgd) | Capital Cost[1] ($M) | O & M Cost[2] (¢/1000 gal) | Total Cost @ 3% (¢/1000 gal) | Total Cost @ 7% (¢/1000 gal) | Total Cost @ 10% (¢/1000 gal) |
|---|---|---|---|---|---|
| 0.024 | 0.011 | 21 | 57 | 71 | 83 |
| 0.087 | 0.012 | 5.5 | 15 | 19 | 22 |
| 0.27 | 0.015 | 1.9 | 5.1 | 6.3 | 7.4 |
| 0.65 | 0.016 | 0.98 | 2.3 | 2.8 | 3.2 |

**LARGE SYSTEMS**

| Design Flow (mgd) | Capital Cost[1] ($M) | O & M Cost[2] (¢/1000 gal) | Total Cost @ 3% (¢/1000 gal) | Total Cost @ 7% (¢/1000 gal) | Total Cost @ 10% (¢/1000 gal) |
|---|---|---|---|---|---|
| 1.8 | 0.04 | 1.4 | 2.5 | 3.0 | 3.4 |
| 4.8 | 0.07 | 0.70 | 1.3 | 1.5 | 1.8 |
| 11 | 0.11 | 0.49 | 0.9 | 1.1 | 1.2 |
| 18 | 0.16 | 0.40 | 0.73 | 0.87 | 0.99 |
| 26 | 0.21 | 0.37 | 0.67 | 0.79 | 0.89 |
| 51 | 0.28 | 0.33 | 0.52 | 0.60 | 0.67 |
| 210 | 0.47 | 0.29 | 0.36 | 0.39 | 0.41 |
| 430 | 0.85 | 0.26 | 0.32 | 0.34 | 0.36 |
| 520 | 0.91 | 0.20 | 0.25 | 0.27 | 0.28 |

[1] 1991 Cost escalated based upon a factor of 1.23 derived from the ENR BCI
[2] 1991 Cost escalated based upon a factor of 1.19 derived from the BLS Chemical and Allied Products Index

**FIGURE 7-3. ALUM COAGULATION / FILTRATION SYSTEMS UPGRADED WITH OZONE / CHLORAMINE DISINFECTION**

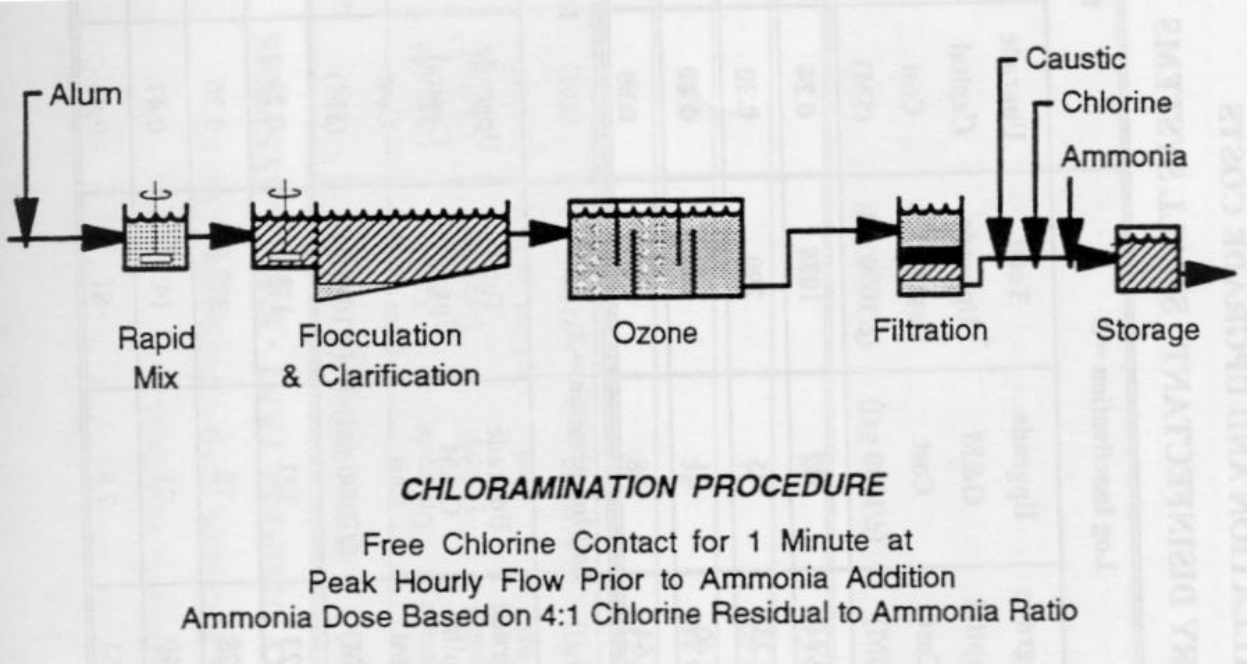


**TABLE 7- 12. ESTIMATED INSTALLATION AND UPGRADE COSTS FOR OZONE AS PRIMARY DISINFECTANT - SMALL SYSTEMS**

| Design Flow (mgd) | Log Inactivation = 1 | | | Log Inactivation = 3 | | | Log Inactivation = 5 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Upgrade Capital Cost ($M) | Upgrade O&M Cost (¢/1000 gal) | Total Upgrade Cost @ 3% (¢/1000 gal) | Upgrade Capital Cost ($M) | Upgrade O&M Cost (¢/1000 gal) | Total Upgrade Cost @ 3% (¢/1000 gal) | Upgrade Capital Cost ($M) | Upgrade O&M Cost (¢/1000 gal) | Total Upgrade Cost @ 3% (¢/1000 gal) |
| 0.024 | 0.22 | 161 | 885 | 0.23 | 322 | 1078 | 0.24 | 644 | 1433 |
| 0.086 | 0.24 | 38 | 222 | 0.28 | 75 | 290 | 0.30 | 150 | 380 |
| 0.27 | 0.29 | 10 | 72 | 0.40 | 21 | 107 | 0.47 | 42 | 143 |
| 0.65 | 0.39 | 3.9 | 35 | 0.64 | 7.8 | 59 | 0.80 | 16 | 80 |
| **Design Flow (mgd)** | **Log Inactivation = 1** | | | **Log Inactivation = 3** | | | **Log Inactivation = 5** | | |
| | **Upgrade Capital Cost ($M)** | **Upgrade O&M Cost (¢/1000 gal)** | **Total Upgrade Cost @ 7% (¢/1000 gal)** | **Upgrade Capital Cost ($M)** | **Upgrade O&M Cost (¢/1000 gal)** | **Total Upgrade Cost @ 7% (¢/1000 gal)** | **Upgrade Capital Cost ($M)** | **Upgrade O&M Cost (¢/1000 gal)** | **Total Upgrade Cost @ 7% (¢/1000 gal)** |
| 0.024 | 0.22 | 161 | 1177 | 0.23 | 322 | 1384 | 0.24 | 644 | 1752 |
| 0.086 | 0.24 | 38 | 297 | 0.28 | 75 | 377 | 0.30 | 150 | 473 |
| 0.27 | 0.29 | 10 | 97 | 0.40 | 21 | 145 | 0.47 | 42 | 183 |
| 0.65 | 0.39 | 3.9 | 48 | 0.64 | 7.8 | 81 | 0.80 | 16 | 106 |

**TABLE 7- 12 (cont)**

**ESTIMATED INSTALLATION AND UPGRADE COSTS FOR OZONE AS PRIMARY DISINFECTANT - SMALL SYSTEMS**

| Design Flow (mgd) | Log Inactivation = 1 | | | Log Inactivation = 3 | | | Log Inactivation = 5 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Upgrade Capital Cost ($M) | Upgrade O&M Cost (¢/1000 gal) | Total Upgrade Cost @ 10% (¢/1000 gal) | Upgrade Capital Cost ($M) | Upgrade O&M Cost (¢/1000 gal) | Total Upgrade Cost @ 10% (¢/1000 gal) | Upgrade Capital Cost ($M) | Upgrade O&M Cost (¢/1000 gal) | Total Upgrade Cost @ 10% (¢/1000 gal) |
| 0.024 | 0.22 | 161 | 1442 | 0.23 | 322 | 1661 | 0.24 | 644 | 2023 |
| 0.086 | 0.24 | 38 | 361 | 0.28 | 75 | 445 | 0.30 | 150 | 551 |
| 0.27 | 0.29 | 10 | 119 | 0.40 | 21 | 170 | 0.47 | 42 | 217 |
| 0.65 | 0.39 | 3.9 | 59 | 0.64 | 7.8 | 98 | 0.80 | 16 | 128 |

**TABLE 7- 12 (cont.)**
**ESTIMATED INSTALLATION AND UPGRADE COSTS FOR OZONE AS PRIMARY DISINFECTANT - LARGE SYSTEMS**

| Design Flow (mgd) | Log Inactivation = 1 | | | Log Inactivation = 3 | | | Log Inactivation = 5 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Upgrade Capital Cost ($M) | Upgrade O&M Cost (¢/1000 gal) | Total Upgrade Cost @3% (¢/1000 gal) | Upgrade Capital Cost ($M) | Upgrade O&M Cost (¢/1000 gal) | Total Upgrade Cost @ 3% (¢/1000 gal) | Upgrade Capital Cost ($M) | Upgrade O&M Cost (¢/1000 gal) | Total Upgrade Cost @ 3% (¢/1000 gal) |
| 1.8 | 0.89 | 1.8 | 25 | 1.5 | 4.4 | 44 | 1.4 | 6.3 | 43 |
| 4.8 | 1.5 | 1.8 | 15 | 1.9 | 4.4 | 21 | 2.0 | 6.3 | 24 |
| 11 | 1.9 | 1.8 | 9.0 | 2.6 | 4.4 | 14 | 2.8 | 6.3 | 17 |
| 18 | 2.4 | 1.8 | 7.0 | 3.0 | 4.4 | 11 | 3.7 | 6.3 | 14 |
| 26 | 2.6 | 1.8 | 5.5 | 3.9 | 4.4 | 9.9 | 4.8 | 6.3 | 13 |
| 51 | 3.8 | 1.8 | 4.4 | 6.2 | 4.4 | 8.6 | 7.4 | 6.3 | 11 |
| 210 | 9.2 | 1.8 | 3.2 | 18 | 4.4 | 7.2 | 24 | 6.3 | 10 |
| 430 | 16.5 | 1.8 | 2.9 | 35 | 4.4 | 6.8 | 47 | 6.3 | 9.5 |
| 520 | 20 | 1.8 | 2.9 | 42 | 4.4 | 6.6 | 57 | 6.3 | 9.3 |

**TABLE 7- 12 (cont.)**
**ESTIMATED INSTALLATION AND UPGRADE COSTS FOR OZONE AS PRIMARY DISINFECTANT - LARGE SYSTEMS**

| Design Flow (mgd) | Log Inactivation = 1 | | | Log Inactivation = 3 | | | Log Inactivation = 5 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Upgrade Capital Cost ($M) | Upgrade O&M Cost (¢/1000 gal) | Total Upgrade Cost @ 7% (¢/1000 gal) | Upgrade Capital Cost ($M) | Upgrade O&M Cost (¢/1000 gal) | Total Upgrade Cost @ 7% (¢/1000 gal) | Upgrade Capital Cost ($M) | Upgrade O&M Cost (¢/1000 gal) | Total Upgrade Cost @ 7% (¢/1000 gal) |
| 1.8 | 0.89 | 1.8 | 35 | 1.5 | 4.4 | 58 | 1.4 | 6.3 | 56 |
| 4.8 | 1.5 | 1.8 | 21 | 1.9 | 4.4 | 28 | 2.0 | 6.3 | 31 |
| 11 | 1.9 | 1.8 | 12 | 2.6 | 4.4 | 17 | 2.8 | 6.3 | 21 |
| 18 | 2.4 | 1.8 | 8.9 | 3.0 | 4.4 | 13 | 3.7 | 6.3 | 17 |
| 26 | 2.6 | 1.8 | 7.2 | 3.9 | 4.4 | 12 | 4.8 | 6.3 | 15 |
| 51 | 3.8 | 1.8 | 5.6 | 6.2 | 4.4 | 10 | 7.4 | 6.3 | 13 |
| 210 | 9.2 | 1.8 | 4.0 | 18 | 4.4 | 7.8 | 24 | 6.3 | 11 |
| 430 | 16.5 | 1.8 | 3.6 | 35 | 4.4 | 7.4 | 47 | 6.3 | 11 |
| 520 | 20 | 1.8 | 3.4 | 42 | 4.4 | 6.1 | 57 | 6.3 | 10 |

**TABLE 7- 12 (cont.)**
**ESTIMATED INSTALLATION AND UPGRADE COSTS**
**FOR OZONE AS PRIMARY DISINFECTANT**
**LARGE SYSTEMS**

| Design Flow (mgd) | Log Inactivation = 1 | | | Log Inactivation = 3 | | | Log Inactivation = 5 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Upgrade Capital Cost ($M) | Upgrade O&M Cost (¢/1000 gal) | Total Upgrade Cost @ 10% (¢/1000 gal) | Upgrade Capital Cost ($M) | Upgrade O&M Cost (¢/1000 gal) | Total Upgrade Cost @ 10% (¢/1000 gal) | Upgrade Capital Cost ($M) | Upgrade O&M Cost (¢/1000 gal) | Total Upgrade Cost @ 10% (¢/1000 gal) |
| 1.8 | 0.89 | 1.8 | 43 | 1.5 | 4.4 | 72 | 1.4 | 6.3 | 69 |
| 4.8 | 1.5 | 1.8 | 26 | 1.9 | 4.4 | 34 | 2.0 | 6.3 | 37 |
| 11 | 1.9 | 1.8 | 14 | 2.6 | 4.4 | 21 | 2.8 | 6.3 | 24 |
| 18 | 2.4 | 1.8 | 11 | 3.0 | 4.4 | 15 | 3.7 | 6.3 | 20 |
| 26 | 2.6 | 1.8 | 8.4 | 3.9 | 4.4 | 14 | 4.8 | 6.3 | 18 |
| 51 | 3.8 | 1.8 | 6.5 | 6.2 | 4.4 | 11 | 7.4 | 6.3 | 15 |
| 210 | 9.2 | 1.8 | 4.5 | 18 | 4.4 | 8.7 | 24 | 6.3 | 12 |
| 430 | 16 | 1.8 | 4.0 | 35 | 4.4 | 8.2 | 47 | 6.3 | 12 |
| 520 | 20 | 1.8 | 3.6 | 42 | 4.4 | 8.2 | 57 | 6.3 | 10 |

**TABLE 7-13 ESTIMATED INSTALLATION AND UPGRADE COSTS FOR CHLORINE DIOXIDE AS PRIMARY DISINFECTANT (MANUAL GENERATOR) SMALL SYSTEMS**

| Design Flow (mgd) | Log Inactivation = 1 | | | Log Inactivation = 3 | | | Log Inactivation = 5 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 3% (¢/1000gal) | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 3% (¢/1000gal) | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 3% (¢/1000gal) |
| 0.024 | 0.10 | 1929 | 2258 | 0.10 | 1934 | 2263 | 0.13 | 1934 | 2362 |
| 0.087 | 0.10 | 452 | 529 | 0.10 | 456 | 533 | 0.15 | 456 | 571 |
| 0.27 | 0.10 | 128 | 149 | 0.10 | 132 | 153 | 0.22 | 132 | 179 |
| 0.65 | 0.10 | 49 | 57 | 0.10 | 52 | 60 | 0.28 | 52 | 74 |
| **Design Flow (mgd)** | **Log Inactivation = 1** | | | **Log Inactivation = 3** | | | **Log Inactivation = 5** | | |
| | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 7% (¢/1000gal) | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 7% (¢/1000gal) | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 7% (¢/1000gal) |
| 0.024 | 0.10 | 1929 | 2391 | 0.10 | 1934 | 2396 | 0.13 | 1934 | 2534 |
| 0.087 | 0.10 | 452 | 560 | 0.10 | 456 | 564 | 0.15 | 456 | 618 |
| 0.27 | 0.10 | 128 | 158 | 0.10 | 132 | 162 | 0.22 | 132 | 198 |
| 0.65 | 0.10 | 49 | 60 | 0.10 | 52 | 63 | 0.28 | 52 | 63 |

**TABLE 7-13 (cont)**
**ESTIMATED INSTALLATION AND UPGRADE COSTS**
**FOR CHLORINE DIOXIDE AS PRIMARY DISINFECTANT**
**(MANUAL GENERATOR)**
**SMALL SYSTEMS**

| Design Flow (mgd) | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 10 % (¢/1000gal) | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 10% (¢/1000gal) | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 10% (¢/1000gal) |
|---|---|---|---|---|---|---|---|---|---|
| 0.024 | 0.10 | 1929 | 2455 | 0.10 | 1934 | 2460 | 0.13 | 1934 | 2659 |
| 0.087 | 0.10 | 452 | 575 | 0.10 | 456 | 579 | 0.15 | 456 | 654 |
| 0.27 | 0.10 | 128 | 162 | 0.10 | 132 | 166 | 0.22 | 132 | 215 |
| 0.65 | 0.10 | 49 | 62 | 0.10 | 52 | 65 | 0.28 | 52 | 91 |

**TABLE 7-13 (cont.)**
**ESTIMATED INSTALLATION AND UPGRADE COSTS**
**FOR CHLORINE DIOXIDE AS PRIMARY DISINFECTANT**
**(AUTOMATIC GENERATOR)**
**SMALL SYSTEMS**

| Design Flow (mgd) | Log Inactivation = 1 | | | Log Inactivation = 3 | | | Log Inactivation = 5 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 3% (¢/1000gal) | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 3% (¢/1000gal) | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 3% (¢/1000gal) |
| 0.024 | 0.33 | 1548 | 2633 | 0.33 | 1552 | 2637 | 0.37 | 1552 | 2769 |
| 0.087 | 0.33 | 364 | 617 | 0.33 | 367 | 620 | 0.39 | 367 | 666 |
| 0.27 | 0.33 | 104 | 175 | 0.33 | 107 | 178 | 0.47 | 107 | 208 |
| 0.65 | 0.33 | 40 | 66 | 0.33 | 43 | 69 | 0.52 | 43 | 85 |
| **Design Flow (mgd)** | **Log Inactivation = 1** | | | **Log Inactivation = 3** | | | **Log Inactivation = 5** | | |
| | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 7% (¢/1000gal) | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 7% (¢/1000gal) | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 7% (¢/1000gal) |
| 0.024 | 0.33 | 1548 | 3072 | 0.33 | 1552 | 3076 | 0.37 | 1552 | 3261 |
| 0.087 | 0.33 | 364 | 720 | 0.33 | 367 | 723 | 0.39 | 367 | 787 |
| 0.27 | 0.33 | 104 | 203 | 0.33 | 107 | 206 | 0.47 | 107 | 248 |
| 0.65 | 0.33 | 40 | 77 | 0.33 | 43 | 80 | 0.52 | 43 | 101 |

**TABLE 7-13 (cont.)**
**ESTIMATED INSTALLATION AND UPGRADE COSTS**
**FOR CHLORINE DIOXIDE AS PRIMARY DISINFECTANT**
**(AUTOMATIC GENERATOR)**
**SMALL SYSTEMS**

| Design Flow (mgd) | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 10% (¢/1000gal) | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 10% (¢/1000gal) | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 10% (¢/1000gal) |
|---|---|---|---|---|---|---|---|---|---|
| 0.024 | 0.33 | 1548 | 3459 | 0.33 | 1552 | 3462 | 0.37 | 1552 | 3662 |
| 0.087 | 0.33 | 364 | 810 | 0.33 | 367 | 813 | 0.39 | 367 | 888 |
| 0.27 | 0.33 | 104 | 228 | 0.33 | 107 | 231 | 0.47 | 107 | 280 |
| 0.65 | 0.33 | 40 | 87 | 0.33 | 43 | 90 | 0.52 | 43 | 115 |

(1) Cost were adjusted to account for an increase in basin contact time from the information reported in Table 7-10

## TABLE 7-13 (cont.) ESTIMATED INSTALLATION AND UPGRADE COSTS FOR CHLORINE DIOXIDE AS PRIMARY DISINFECTANT (MANUAL GENERATOR)

### LARGE SYSTEMS

| Design Flow (mgd) | Log Inactivation = 1 | | | Log Inactivation = 3 | | | Log Inactivation = 5 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Upgrade Capital Cost[(1)] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 3% (¢/1000gal) | Upgrade Capital Cost[(1)] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 3% (¢/1000gal) | Upgrade Capital Cost[(1)] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 3% (¢/1000gal) |
| 1.8 | 0.10 | 18 | 20.6 | 0.10 | 20 | 22.6 | 0.50 | 20 | 33 |
| 4.8 | 0.10 | 7.4 | 8.3 | 0.10 | 9.6 | 10.5 | 0.75 | 9.6 | 16 |
| 11 | 0.10 | 4.3 | 4.7 | 0.10 | 6.5 | 6.9 | 1.3 | 6.5 | 11.3 |
| 18 | 0.20 | 3.8 | 4.2 | 0.20 | 5.8 | 6.2 | 3.7 | 5.8 | 13.5 |
| 26 | 0.20 | 3.2 | 3.5 | 0.20 | 5.1 | 5.4 | 5.1 | 5.1 | 12.3 |
| 51 | 0.28 | 2.4 | 2.6 | 0.28 | 4.6 | 4.8 | 9.4 | 4.6 | 11 |
| 210 | 0.29 | 1.8 | 1.8 | 0.29 | 3.6 | 3.6 | 36 | 3.6 | 9.1 |
| 430 | 0.35 | 1.6 | 1.6 | 0.35 | 3.2 | 3.2 | 73 | 3.2 | 8.2 |
| 520 | 0.35 | 1.4 | 1.4 | 0.35 | 2.4 | 2.4 | 89 | 2. | 7.1 |

**TABLE 7-13 (cont.)**
**ESTIMATED INSTALLATION AND UPGRADE COSTS**
**FOR CHLORINE DIOXIDE AS PRIMARY DISINFECTANT**
**(MANUAL GENERATOR)**
**LARGE SYSTEMS**

| | Log Inactivation = 1 | | | Log Inactivation = 3 | | | Log Inactivation = 5 | | |
|---|---|---|---|---|---|---|---|---|---|
| **Design Flow (mgd)** | **Upgrade Capital Cost[(1)] ($M)** | **Upgrade O&M Cost (¢/1000gal)** | **Total Upgrade Cost @ 7% (¢/1000gal)** | **Upgrade Capital Cost[(1)] ($M)** | **Upgrade O&M Cost (¢/1000gal)** | **Total Upgrade Cost @ 7% (¢/1000gal)** | **Upgrade Capital Cost[(1)] ($M)** | **Upgrade O&M Cost (¢/1000gal)** | **Total Upgrade Cost @ 7% (¢/1000gal)** |
| 1.8 | 0.10 | 18 | 22 | 0.10 | 20 | 24 | 0.50 | 20 | 38 |
| 4.8 | 0.10 | 7.4 | 8.2 | 0.10 | 9.6 | 11 | 0.75 | 9.6 | 19 |
| 11 | 0.10 | 4.3 | 4.5 | 0.10 | 6.5 | 6.5 | 1.3 | 6.5 | 13 |
| 18 | 0.20 | 3.8 | 4.6 | 0.20 | 5.8 | 6.6 | 3.7 | 5.8 | 17 |
| 26 | 0.20 | 3.2 | 3.4 | 0.20 | 5.1 | 5.4 | 5.1 | 5.1 | 15 |
| 51 | 0.28 | 2.4 | 2.3 | 0.28 | 4.6 | 4.3 | 9.4 | 4.6 | 13 |
| 210 | 0.29 | 1.8 | 2.1 | 0.29 | 3.6 | 4.1 | 36 | 3.6 | 12 |
| 430 | 0.35 | 1.6 | 2.0 | 0.35 | 3.2 | 3.0 | 73 | 3.2 | 10 |
| 520 | 0.35 | 1.4 | 1.4 | 0.35 | 2.4 | 2.5 | 89 | 2.4 | 9.0 |

**TABLE 7-13 (cont.)**
**ESTIMATED INSTALLATION AND UPGRADE COSTS**
**FOR CHLORINE DIOXIDE AS PRIMARY DISINFECTANT**
**(MANUAL GENERATOR)**

**LARGE SYSTEMS**

| Design Flow (mgd) | Log Inactivation = 1 | | | Log Inactivation = 3 | | | Log Inactivation = 5 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Upgrade Capital Cost[(1)] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 10% (¢/1000gal) | Upgrade Capital Cost[(1)] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 10% (¢/1000gal) | Upgrade Capital Cost[(1)] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 10% (¢/1000gal) |
| 1.8 | 0.10 | 18 | 22 | 0.10 | 20 | 25 | 0.50 | 20 | 43 |
| 4.8 | 0.10 | 7.4 | 8.8 | 0.10 | 10 | 11 | 0.75 | 10 | 21 |
| 11 | 0.10 | 4.3 | 4.9 | 0.10 | 6.5 | 7.1 | 1.3 | 6.5 | 15 |
| 18 | 0.20 | 3.8 | 4.5 | 0.20 | 5.8 | 6.5 | 3.7 | 5.8 | 19 |
| 26 | 0.20 | 3.2 | 3.7 | 0.20 | 5.1 | 5.6 | 5.1 | 5.1 | 18 |
| 51 | 0.28 | 2.4 | 2.8 | 0.28 | 4.3 | 4.6 | 9.4 | 4.3 | 15 |
| 210 | 0.29 | 1.8 | 1.9 | 0.29 | 3.5 | 3.6 | 36 | 3.5 | 13 |
| 430 | 0.35 | 1.6 | 1.7 | 0.35 | 3.2 | 3.2 | 73 | 3.2 | 12 |
| 520 | 0.35 | 1.4 | 1.4 | 0.35 | 2.4 | 2.5 | 89 | 2.4 | 11 |

(1) Cost were adjusted to account for an increase in basin contact time from the information reported in Table 7-10

**TABLE 7-13 (cont.)**
**ESTIMATED INSTALLATION AND UPGRADE COSTS**
**FOR CHLORINE DIOXIDE AS PRIMARY DISINFECTANT**
**(AUTOMATIC GENERATOR)**

**LARGE SYSTEMS**

| Design Flow (mgd) | Log Inactivation = 1 | | | Log Inactivation = 3 | | | Log Inactivation = 5 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 3% (¢/1000gal) | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 3% (¢/1000gal) | Upgrade Capital Cost[1] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 3% (¢/1000gal) |
| 1.8 | 0.33 | 15 | 24 | 0.33 | 17 | 26 | 0.74 | 17 | 37 |
| 4.8 | 0.33 | 6.3 | 9.2 | 0.33 | 8.6 | 12 | 0.99 | 8.6 | 17 |
| 11 | 0.33 | 3.9 | 5.1 | 0.33 | 6.0 | 7.2 | 1.5 | 6.0 | 12 |
| 18 | 0.68 | 3.3 | 4.7 | 0.68 | 5.3 | 6.7 | 4.2 | 5.3 | 14 |
| 26 | 0.68 | 2.9 | 3.9 | 0.68 | 4.8 | 5.8 | 5.6 | 4.8 | 13 |
| 51 | 0.76 | 2.3 | 2.8 | 0.76 | 4.1 | 4.6 | 10 | 4.1 | 11 |
| 210 | 0.77 | 1.8 | 1.9 | 0.77 | 3.5 | 3.6 | 37 | 3.5 | 9.2 |
| 430 | 0.83 | 1.6 | 1.7 | 0.83 | 3.1 | 3.2 | 74 | 3.1 | 8.2 |
| 520 | 0.91 | 1.3 | 1.4 | 0.91 | 2.5 | 2.6 | 90 | 2.5 | 7.2 |

**TABLE 7-13 (cont.)**
**ESTIMATED INSTALLATION AND UPGRADE COSTS**
**FOR CHLORINE DIOXIDE AS PRIMARY DISINFECTANT**
**(AUTOMATIC GENERATOR)**

**LARGE SYSTEMS**

| Design Flow (mgd) | Log Inactivation = 1 | | | Log Inactivation = 3 | | | Log Inactivation = 5 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Upgrade Capital Cost[(1)] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 7% (¢/1000gal) | Upgrade Capital Cost[(1)] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 7% (¢/1000gal) | Upgrade Capital Cost[(1)] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 7% (¢/1000gal) |
| 1.8 | 0.33 | 15 | 27 | 0.33 | 17 | 29 | 0.74 | 17 | 44 |
| 4.8 | 0.33 | 6.3 | 10 | 0.33 | 8.6 | 13 | 0.99 | 8.6 | 21 |
| 11 | 0.33 | 3.9 | 5.7 | 0.33 | 6.0 | 7.7 | 1.5 | 6.0 | 14 |
| 18 | 0.68 | 3.3 | 5.0 | 0.68 | 5.3 | 7.0 | 4.2 | 5.3 | 17 |
| 26 | 0.68 | 2.9 | 4.4 | 0.68 | 4.8 | 6.4 | 5.6 | 4.8 | 16 |
| 51 | 0.76 | 2.3 | 2.7 | 0.76 | 4.1 | 4.7 | 10 | 4.1 | 13 |
| 210 | 0.77 | 1.8 | 2.2 | 0.77 | 3.5 | 4.2 | 37 | 3.5 | 12 |
| 430 | 0.83 | 1.6 | 2.1 | 0.83 | 3.1 | 3.1 | 74 | 3.1 | 10 |
| 520 | 0.91 | 1.3 | 1.4 | 0.91 | 2.5 | 2.6 | 90 | 2.5 | 9.1 |

(1) Cost were adjusted to account for an increase in basin contact time from the information reported in Table 7-10

**TABLE 7-13 (cont.)**
**ESTIMATED INSTALLATION AND UPGRADE COSTS**
**FOR CHLORINE DIOXIDE AS PRIMARY DISINFECTANT**
**(AUTOMATIC GENERATOR)**

**LARGE SYSTEMS**

| Design Flow (mgd) | Log Inactivation = 1 | | | Log Inactivation = 3 | | | Log Inactivation = 5 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Upgrade Capital Cost[(1)] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 10% (¢/1000gal) | Upgrade Capital Cost[(1)] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 10% (¢/1000gal) | Upgrade Capital Cost[(1)] ($M) | Upgrade O&M Cost (¢/1000gal) | Total Upgrade Cost @ 10% (¢/1000gal) |
| 1.8 | 0.33 | 15 | 30 | 0.33 | 17 | 32 | 0.74 | 17 | 51 |
| 4.8 | 0.33 | 6.3 | 11 | 0.33 | 8.6 | 14 | 0.99 | 8.6 | 24 |
| 11 | 0.33 | 3.9 | 6.1 | 0.33 | 6.0 | 8.1 | 1.5 | 6.0 | 16 |
| 18 | 0.68 | 3.3 | 5.5 | 0.68 | 5.3 | 7.5 | 4.2 | 5.3 | 21 |
| 26 | 0.68 | 2.9 | 4.7 | 0.68 | 4.8 | 6.7 | 5.6 | 4.8 | 19 |
| 51 | 0.76 | 2.3 | 2.9 | 0.76 | 4.1 | 4.9 | 10 | 4.1 | 16 |
| 210 | 0.77 | 1.8 | 2.2 | 0.77 | 3.5 | 4.2 | 37 | 3.5 | 13 |
| 430 | 0.83 | 1.6 | 2.1 | 0.83 | 3.1 | 3.1 | 74 | 3.1 | 12 |
| 520 | 0.91 | 1.3 | 1.4 | 0.91 | 2.5 | 2.7 | 90 | 2.5 | 11 |

(1) Cost were adjusted to account for an increase in basin contact time from the information reported in Table 7-10

## B.2 References


1. USEPA. 1998a. Technologies and Costs for Control of Disinfection By-Products. Washington, DC.

2. USEPA. 1998b. *Regulatory Impact Analysis for the Stage 1 Disinfectant/Disinfection Byproduct Rule*. Prepared by Science Applications International Corporation for the USEPA, Office of Ground Water and Drinking Water, Washington, DC.

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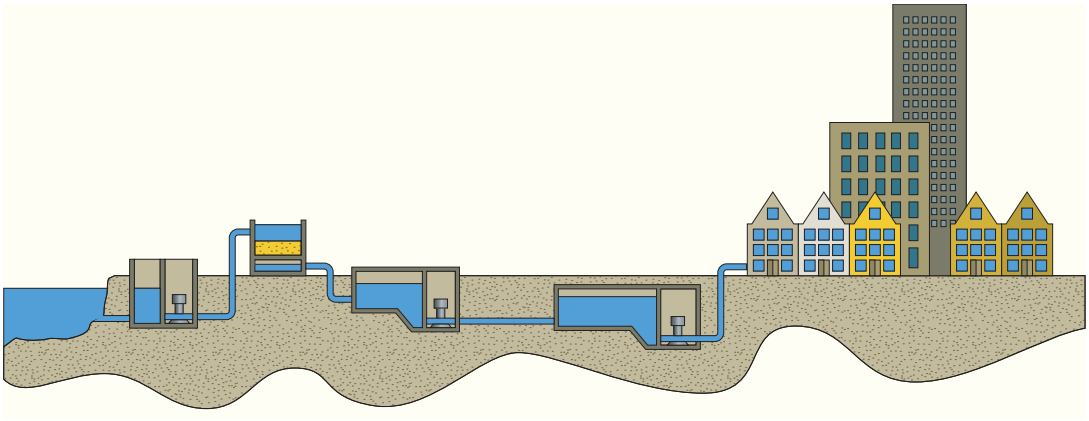
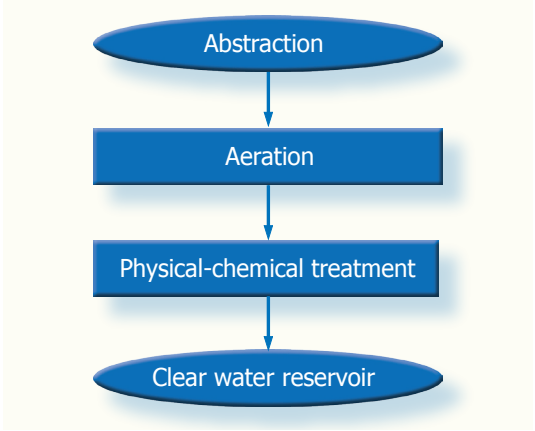
# الملحق الرابع

# Annex 4

WATER TREATMENT

# Water treatment schemes





## Framework

This module represents a short introduction to treatment schemes used for the production of drinking water.

## Contents

This module has the following contents:

1. Introduction
2. Groundwater
   - 2.1 Types of groundwater
   - 2.2 Aerobic groundwater (phreatic)
   - 2.3 Slightly anaerobic groundwater
   - 2.4 Deep anaerobic groundwater
3. Riverbank groundwater
   - 3.1 Types of riverland groundwater
   - 3.2 Riverland groundwater
   - 3.3 Riverbank filtrate
4. Surface water with direct treatment
   - 4.1 Historic development
   - 4.2 Contemporary treatment
   - 4.3 Future treatment
5. Surface water with infiltration
   - 5.1 Surface water with open infiltration
   - 5.2 Surface water with deep infiltration
   - 5.3 Pretreatment
   - 5.4 Infiltration
   - 5.5 Final treatment of infiltrated water

Further reading
Questions and applications
Answers

## Study goals

After having studied this module, you will be able to:

- understand treatment schemes for groundwater and surface water

## 1. Introduction

For personal hygiene and other domestic activities (washing, cleaning, toilet flushing, etc.), it is important to have sufficient water available. In addition, for consumption, water of good quality is essential.

More than 99% of Dutch households receive their water via public water supply.
This water is hygienically reliable, clear, good tasting and of a pleasant temperature. The water is distributed through piped networks under sufficient pressure for even the most remote houses. High quality drinking water is supplied into people's homes at a very "low" price.
The Dutch water supply system is one of the best-known systems in the world.

In public water supply, five components may be distinguished (Figure 1):
- abstraction
- treatment
- transport
- storage
- distribution.

Raw water can be abstracted from groundwater or surface water. For almost every type of water, a corresponding treatment is necessary before it can be supplied as drinking water. During transport, storage and distribution, the quality of the drinking water must not deteriorate below the established standards.

The purpose of placing a low-lying, clear water reservoir after the treatment step is to adjust the flow differences between the treatment plant and

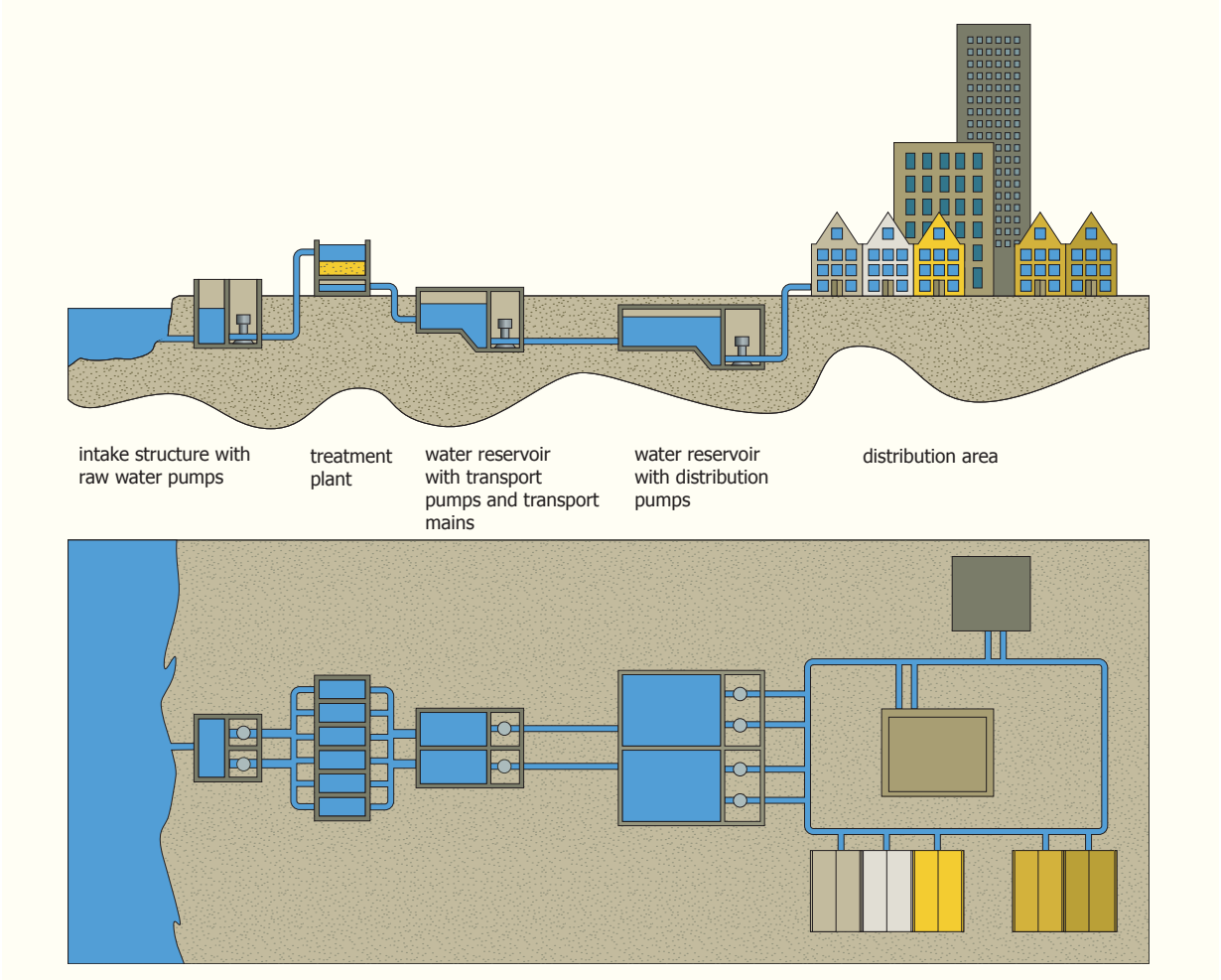



*Figure 1 - Drinking water supply of a city*

the transport pumps. A distribution reservoir at the edge of the city levels off consumption variations between day and night, resulting in a constant flow through the treatment plant and the transport main, and in minimal energy consumption.
The drinking water is distributed continuously under sufficient pressure (>20 meters water column (200 kPa) above street level). This pressure also prevents infiltration of the groundwater into the distribution system, which would result in quality deterioration.

Because drinking water quality is only required for human consumption (5 % of the total production), separate water supplies with different qualities could be considered. The distribution network, however, is the most expensive element of the public water supply. The costs are up to 50 to 70% of the total water price. The savings on treatment costs do not compensate for the extra costs for supply and distribution and the risks to public health (by cross-connections).

Domestic water consumption in the Netherlands is still increasing, despite efforts to encourage consumers to save water (Figure 2).
In addition, the standards for drinking water have become more rigorous and the knowledge about contamination and its effects on public health are increasing.

Important developments are the discovery of harmful by-products that are formed during the chemical disinfection of drinking water.

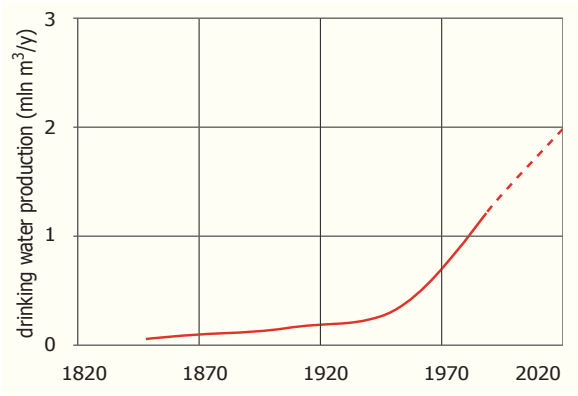



*Figure 2 - Drinking water production in the Netherlands*

Parallel to these developments, the discovery of pesticides in raw water and drinking water (the Bentazon-affair at Amsterdam Water Supply in 1987) has led to changes in opinions about drinking water production.
Additionally, many water companies have set guidelines for the maximum hardness of drinking water. Hard water can cause lime deposits in warm water installations and reduce the formation of foam from soap. Hence, energy and soap consumption increase.

Finally, more rigorous environmental standards have led to better treatment of surface water before it is infiltrated.
Also, backwash water for the filters is more often treated and recirculated.

The above implies that, in the near future, the infrastructure must be expanded and improved.
For civil engineers this creates a great challenge.
The design of the components of a drinking water supply system requires not only knowledge of sani-tary engineering, but also knowledge of other disciplines as well.

Knowledge of water management is inevitable for the choice of the source and the consequences of the abstraction for the environment and other activities (agriculture, navigation).
In addition, there is a need for fluid mechanics and structural engineering to determine the dimensions of pipes, pumps, treatment installations and reservoirs.
Finally, knowledge of chemical engineering and biotechnology is needed to design and optimize treatment processes.

## 2. Groundwater

### *2.1 Types of groundwater*

Groundwater has a near-constant quality. Per location, however, large differences in water composition can be found.
This composition is a result of the natural environment from which the groundwater is

abstracted, and the route that the water has followed to get there.

Three types of groundwater can roughly be distinguished with respect to the treatment in drinking water production:
- aerobic groundwater (phreatic)
- slightly anaerobic groundwater
- deep anaerobic groundwater

The above list implies that, for the treatment of groundwater, the level of oxygen (aerobic, slightly or deep anaerobic) is very important.
The redox potential is a good indicator for this, but this potential is seldom measured in practice. To what type a certain groundwater belongs can be determined from the concentrations of oxygen, iron, and methane.
The three types of groundwater will be further discussed separately, on both their typical characteristics and their treatment schemes. After this the different treatment processes will be described.

## 2.2 *Aerobic groundwater (phreatic)*

Phreatic groundwater has an open groundwater table and is, consequently, connected to the atmosphere. When the organic matter content of the soil is limited, the water does not lose its oxygen (i.e., become anaerobic). As a result, no anaerobic reactions (e.g., iron dissolution) occur in the soil.

In special cases aerobic groundwater meets the requirements for drinking water.
In the Netherlands a couple of groundwater abstraction facilities are located on the Veluwe from which the abstracted water is directly distributed as drinking water. Normally, some treatment is necessary or desired.
Despite the fact that we are dealing with aerobic (i.e., containing oxygen) groundwater, the first treatment step is an aeration phase. Because of this aeration phase, the concentration of oxygen is increased further and the concentration of carbon dioxide is decreased. If the water complies with the legal standards after this treatment step, then the water can be distributed.

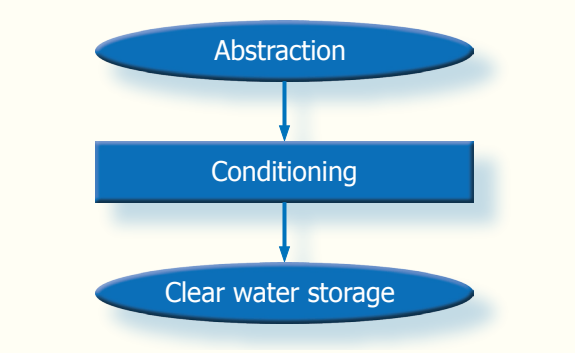



*Figure 3 - Treatment of phreatic aerobic groundwater*

In the case of aerobic phreatic groundwater, only the parameters pH, Ca, SI and $HCO_3^-$ have to be taken into account. The other parameters generally comply with the legal requirements.
Therefore, the treatment scheme of phreatic aerobic groundwater includes, in addition to a possible aeration, conditioning (Figure 3).

**Aggressive water**

When aerobic groundwater is abstracted from sandy soils (no calcium in the underground), the groundwater is often aggressive to limestone.
Because of a number of breakdown processes, carbon dioxide is present in groundwater, and, because the calcium is missing, the concentration of carbon dioxide is higher than the equilibrium concentration of carbon dioxide. The value of the saturation index, SI, is smaller than 0. To make distribution of this water possible, the saturation index has to be increased.

The SI is increased by aerating the water, which removes carbon dioxide. Then the SI may meet the requirements, but the requirements for pH and $HCO_3^-$ buffering are often not met because the concentration of $HCO_3^-$ is too low. When limestone (marble) filtration is applied, the requirements for SI, pH and $HCO_3^-$ buffering are met. During limestone filtration, the aggressive water is filtered through a filter bed consisting of marble grains (limestone) (Figure 4). Because the water is aggressive, it

*Figure 4 - Aeration above limestone filter*

dissolves the marble grains. After some time, the filter bed has to be refilled with new grains.

**Hard water**

Aerobic groundwater, which is abstracted from soils rich in calcium (for example, limestone area of Zuid-Limburg), is often very hard (>3 mmol/l). Because of the biological processes in the soil, the concentration of $CO_2$ can be resulting in substantial dissolution of limestone, forming $Ca^{2+}$ and $HCO_3^-$ in the water. The abstracted water, therefore, is hard.

Groundwater will sometimes be in equilibrium regarding calcium carbonate (limestone). Water that is supersaturated with respect to calcium carbonate cannot be found in nature; because of the long residence time, a possible supersaturation would already have disappeared due to precipitation. When this water is pumped up and comes in contact with air, the carbon dioxide disappears from the water. The carbon dioxide concentration is, after all, larger than the saturation concentration of carbon dioxide in water being in equilibrium with air. Because of the removal of carbon dioxide from the water, the water becomes supersaturated with respect to calcium carbonate (SI > 0).

*Figure 5 - Pellet reactor for softening water in Meersen (Limburg)*

To prevent limestone precipitation in a distribution network or in consumers' washing machines and heaters, and to satisfy the recommendation of a maximum hardness of 1.5 mmol/l, the water is softened. This softening occurs by dosing chemicals (NaOH or $Ca(OH)_2$) into the water in cylindrical reactors with upward flow (Figure 5). These reactors contain small sand grains, which are used as crystallization nuclei on which the $CaCO_3$ precipitates.

The softening installation should be followed by granular media filtration, because possible post-precipitation might occur. After all, the time the water stays in the pellet reactor is short (a couple of minutes), and for the complete process of chemical softening more time is needed. When, after the softening, a granular media filtration phase is executed, post-precipitation takes place in the filter bed. If this filtration phase isn't provided, then the precipitation will take place in the distribution network or in the consumers' household machines.
Alternatively, acid neutralization can be applied.

**Example**

As an example of the change in water quality, the Hoenderloo pumping station on the Veluwe is described. Treatment at the Hoenderloo pumping station consists of aeration/gas transfer followed by limestone filtration. The values of the different parameters in Table 1 are the annual averages.

The pH of the treated water is higher than the raw water, because the water is aerated (removal of

*Table 1 - Quality data of the raw and treated water at the Hoenderloo pumping station (Gelderland)*

| Parameter | Unit | Raw water | Clear water |
|---|---|---|---|
| Temperature | °C | 9.6 | 10 |
| pH | - | 6.1 | 7.8 |
| EGV | mS/m | 9.3 | 14.3 |
| SI | - | -3.4 | -0.3 |
| Turbidity | FTU | - | < 0.1 |
| $Na^+$ | mg/l | 8.1 | 7.9 |
| $K^+$ | mg/l | 1 | 1 |
| $Ca^{2+}$ | mg/l | 8.6 | 22.5 |
| $Mg^{2+}$ | mg/l | 1.6 | 1.6 |
| $Cl^-$ | mg/l | 12 | 12 |
| $HCO_3^-$ | mg/l | 21 | 63 |
| $SO_4^{2-}$ | mg/l | 9 | 10 |
| $NO_3^-$ | mg/l | 2.7 | 2.7 |
| $O_2$ | mg/l | 4.2 | 8 |
| $CH_4$ | mg/l | - | - |
| $CO_2$ | mg/l | 31 | 2 |
| $Fe^{2+}$ | mg/l | 0.06 | 0.03 |
| $Mn^{2+}$ | mg/l | 0.02 | < 0.01 |
| $NH_4^+$ | mg/l | < 0.04 | < 0.04 |
| DOC | mg/l | < 0.2 | < 0.2 |
| E-Coli | n/100 ml | 0 | 0 |
| Bentazon | µg/l | - | - |
| Chloroform | µg/l | - | - |
| Bromate | µg/l | - | - |

$CO_2$) and because the water is filtered through a limestone filter (decrease of the $CO_2$ concentration, increase of the $HCO_3^-$ and the $Ca^{2+}$ concentration). The SI increases under the influence of the lower concentration of $CO_2$, and the higher $HCO_3^-$, and the $Ca^{2+}$ concentrations; the water becomes less aggressive with respect to calcium carbonate.

Because of the use of limestone filtration, the $HCO_3^-$ and the $Ca^{2+}$ concentrations will increase. More ions will get into the water, as a result of which the conductivity (EC) increases.
We can calculate the increase in the $HCO_3^-$ concentration when we assume that all produced $HCO_3^-$ comes from the limestone. For every formed mmol/l $Ca^{2+}$, 2 mmol/l $HCO_3^-$ are produced. At this pumping station the $Ca^{2+}$ -concentration increases with 0.3475 mmol/l because of the limestone filtration, and the concentration of $HCO_3^-$ has to be increased by $2 \cdot 0.3475 = 0.695$ mmol/l. There was 0.34 mmol/l $HCO_3^-$ present in the raw water and thus, there has to be 0.695 + 0.34 = 1.035 mmol/l $HCO_3^-$ in the treated water. This corresponds to 63.1 mg/l.

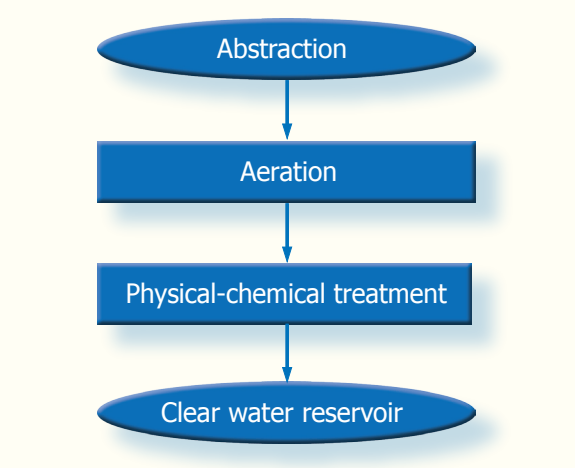



*Figure 6 - Treatment of slightly anaerobic groundwater*

### 2.3 Slightly anaerobic groundwater

Slightly anaerobic groundwater is found when the groundwater is located under a confining layer, and is characterized by the lack of oxygen and the presence of ammonium, iron and manganese.
The treatment of slightly anaerobic groundwater often consists of aeration followed by submerged granular media filtration (Figures 6 and 7).
Aeration is necessary for the addition of oxygen and the removal of carbon dioxide. The oxygen is used for the oxidation of $Fe^{2+}$ to $Fe^{3+}$ (a chemical process), and it is also needed for the oxidation of $NH_4^+$ to $NO_3^-$ and of $Mn^{2+}$ to $MnO_2$.
Aeration is followed by submerged sand filtration. In the filter the oxidized ferric iron reacts with $OH^+$-ions and is transformed into $Fe(OH)_3$-flocs, which are filtered in the sand bed (a physical process).
Manganese undergoes a partly chemical and partly biological transformation, while ammonium is biologically transformed. The transformation

*Figure 7 - Treatment of slightly anaerobic groundwater*

of ammonium is accomplished by the bacteria *Nitrosomonas* and *Nitrobacter*. During this transformation a lot of oxygen is used; per mg/l ammonium, the oxygen consumed is 3.55 mg/l. Also, a lot of nitrate is formed; per mg/l ammonium, 3.44 mg/l nitrate is produced.

As a result of the biological transformation of ammonium and manganese and the physical removal of the iron hydroxide flocs, the pore volume between the sand grains decreases, because the pores are filled by either bacteria or by flocs and deposits. The result of this is the increase in the hydraulic resistance of the water when flowing through the filter bed. When this resistance becomes too large, the filter should be backwashed.

**Example**

As an example of the change in water quality of slightly anaerobic water, the pumping station at the Zutphenseweg will be described. The treatment at the Zutphenseweg pumping station consists of aeration/gas transfer followed by sand filtration and a second aeration. The values of the different parameters in Table 2 are the average values over a year. As a result of aeration the concentration of $CO_2$ will decrease and the pH of the water will increase. Because of aeration the concentration of oxygen will increase to a value near the saturation value (ca. 10 mg/l). The concentration of $Fe^{2+}$, $Mn^{2+}$ and $NH_4^+$ will decrease due to the influence of oxidation and biological transformations. The nitrate content will increase, because the ammonium is transformed into nitrate. Since the decrease in ammonium is approximately 0.8 mg/l, the nitrate content should increase circa 2.7 mg/l. The oxygen consumption is approximately 2.8 mg/l. To get a high oxygen content, post-aeration is used.

*Table 2 - Quality data of the raw and treated water at Zutphenseweg pumping station (Overijssel)*

| Parameter | Unit | Raw water | Clear water |
|---|---|---|---|
| Temperature | °C | 13.1 | 13.1 |
| pH | - | 7.7 | 7.9 |
| EGV | mS/m | 58 | 58 |
| SI | - | -0.1 | 0.1 |
| Turbidity | FTU | - | < 0.1 |
| $Na^+$ | mg/l | 75 | 75 |
| $K^+$ | mg/l | 6.7 | 6.7 |
| $Ca^{2+}$ | mg/l | 47 | 46 |
| $Mg^{2+}$ | mg/l | 7.8 | 8 |
| $Cl^-$ | mg/l | 108 | 110 |
| $HCO_3^-$ | mg/l | 185 | 177 |
| $SO_4^{2-}$ | mg/l | < 1 | < 1 |
| $NO_3^-$ | mg/l | < 0.1 | 2.8 |
| $O_2$ | mg/l | 0.4 | 9.5 |
| $CH_4$ | mg/l | - | - |
| $CO_2$ | mg/l | 7 | 4 |
| $Fe^{2+}$ | mg/l | 0.39 | 0.03 |
| $Mn^{2+}$ | mg/l | 0.03 | < 0.01 |
| $NH_4^+$ | mg/l | 0.82 | < 0.04 |
| DOC | mg/l | 2 | 1,7 |
| E-Coli | n/100 ml | 0 | 0 |
| Bentazon | µg/l | - | - |
| Chloroform | µg/l | - | - |
| Bromate | µg/l | - | - |

## 2.4 Deep anaerobic groundwater

Deep anaerobic groundwater is found when the water is abstracted under a confining layer and no oxygen is present in the water. Furthermore, there is also no nitrate present and organic material is broken down with sulfate as an oxidant. Iron,

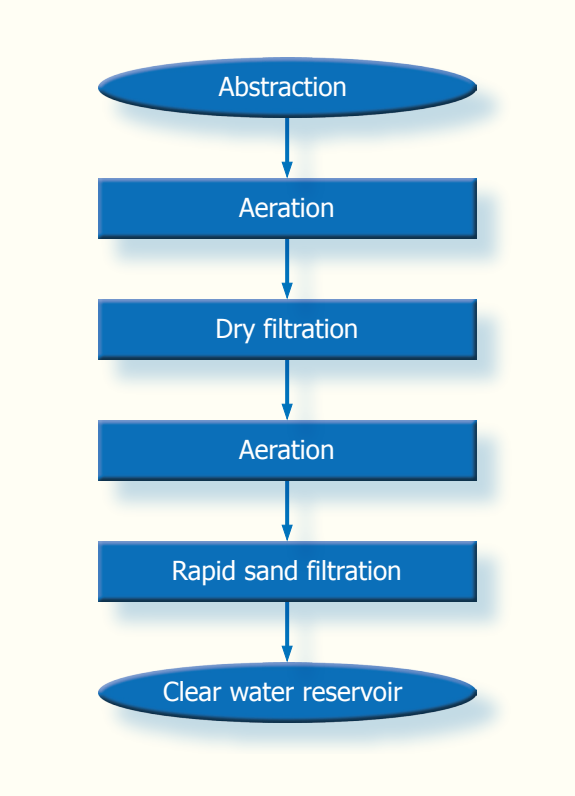



*Figure 8 - Treatment of deep anaerobic groundwater*

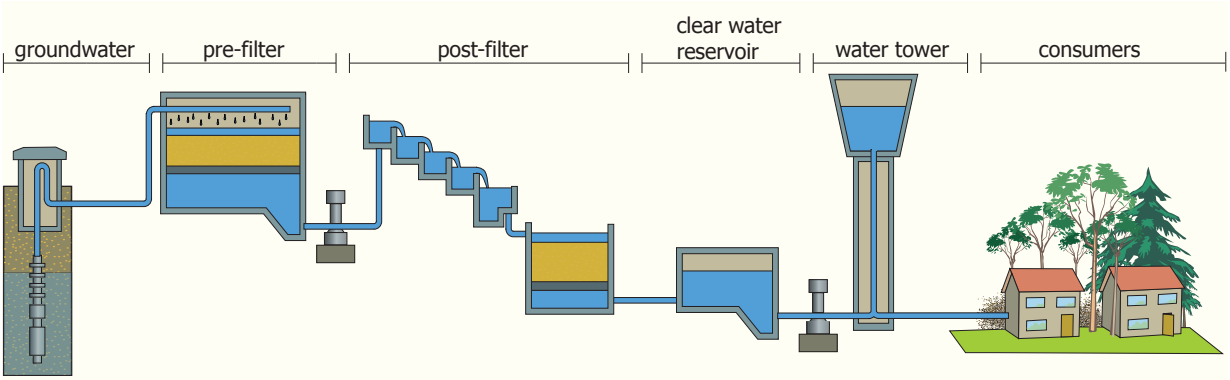



*Figure 9 - Treatment of groundwater with double aeration/filtration*

manganese and especially ammonium are present in high concentrations, while hydrogen sulfide and methane are also present in the groundwater.

During the removal of ammonium, a lot of oxygen is used. When the ammonium content is larger than 3 mg/l, the amount of oxygen necessary for the removal of the ammonium is larger than the total amount of oxygen, which can be dissolved in water (saturation concentration). To prevent anaerobic conditions in the last filter, double submerged filtration or dry filtration followed by submerged filtration is used during groundwater treatment with a high amount of ammonium.

Dry filtration is followed by submerged filtration because in a dry filter, the breakthrough of particles may occur. When these materials pass through the dry filter, they are filtered in the submerged filter and do not show up in the drinking water.

An aeration phase is present before every filtration step, so the oxygen concentration is high before the water enters the filter and the carbon dioxide is removed (Figures 8 and 9). A dry filter is a filter filled with sand grains with a diameter between 0.8 and 4 mm. A layer of water is not present in the filter, like with submerged filtration. In the dry filter the water flows down past the grains, at the same time air is flowing with the water. The oxygen in the air replenishes the oxygen in the water, which is used by bacteria. In this way more than 3 mg/l of ammonium can be transformed without anaerobic results in the filter.

**Example**

As an example of the change in water quality of deep anaerobic groundwater, the St. Jansklooster pumping station is described. The treatment at this pumping station consists of aeration, dry filtration, aeration and submerged filtration. The values of the different parameters in Table 3 are the average values over a year.

As a result of the aeration phases, the amount of carbon dioxide will decrease and the pH will increase. Furthermore, the amount of oxygen will

*Table 3 - Quality data of the raw and treated water at St. Jansklooster pumping station (Overijssel)*

| Parameter | Unit | Raw water | Clear water |
|---|---|---|---|
| Temperature | °C | 10.5 | 10.5 |
| pH | - | 6.9 | 7.6 |
| EGV | mS/m | 51 | 48 |
| SI | - | -0.4 | 0.2 |
| Turbidity | FTU | - | < 0.1 |
| $Na^+$ | mg/l | 23 | 21 |
| $K^+$ | mg/l | 3 | 3 |
| $Ca^{2+}$ | mg/l | 82 | 77 |
| $Mg^{2+}$ | mg/l | 5.2 | 6.3 |
| $Cl^-$ | mg/l | 41 | 41 |
| $HCO_3^-$ | mg/l | 267 | 241 |
| $SO_4^{2-}$ | mg/l | 18 | 21 |
| $NO_3^-$ | mg/l | 0.07 | 1.6 |
| $O_2$ | mg/l | 0 | 10.7 |
| $CH_4$ | mg/l | 2 | < 0.05 |
| $CO_2$ | mg/l | 63 | 11 |
| $Fe^{2+}$ | mg/l | 8.8 | 0.04 |
| $Mn^{2+}$ | mg/l | 0.3 | < 0.01 |
| $NH_4^+$ | mg/l | 2.2 | < 0.01 |
| DOC | mg/l | 7 | 6 |
| E-Coli | n/100 ml | 0 | 0 |
| Bentazon | µg/l | - | - |
| Chloroform | µg/l | - | - |
| Bromate | µg/l | - | - |

increase. The concentration of $Fe^{2+}$, $Mn^{2+}$ and $NH_4^+$ will decrease because of chemical and biological transformations; the amount of nitrate, on the other hand, will increase. The concentration of nitrate increases less than the theoretical calculation.

## 3 Riverbank groundwater

### *3.1 Types of riverbank groundwater*

Riverbank filtration is groundwater abstracted directly adjacent to surface water, usually from a river. The abstraction takes place in such a way that the abstracted water consists mostly of surface water. This surface water is infiltrated into the soil via the riverbank on the river bottom. In this way, a mixture of surface water and natural groundwater is abstracted.

The residence time of the infiltrated surface water in the soil can be several years. In this case we call it riverbank groundwater. This groundwater has the characteristics of groundwater, but the chemical composition also reveals surface water. In the Netherlands such abstractions are found along the Lek and the IJssel. The distance between abstraction wells and the river vary between 200 and 1,000 m.

In Germany most abstraction wells are placed much closer to the river (Figure 10). Residence times of several weeks are common. It is then called riverbank filtrate. It is clear that the existence of surface water in such cases is easier to recognize. A well-defined boundary between riverbank groundwater and riverbank filtrate doesn't exist.

*Figure 10 - Riverbank groundwater well in Germany*

### *3.2 Riverbank groundwater*

The treatment of riverbank groundwater has many similarities to the treatment of slightly anaerobic groundwater. Riverbank groundwater is actually for a large part "natural" groundwater. The other part is surface water that has some of the characteristics of groundwater due to a long residence time in the soil.

The treatment scheme for riverbank groundwater is shown in Figure 11. Depending on the soil composition, higher concentrations of iron,

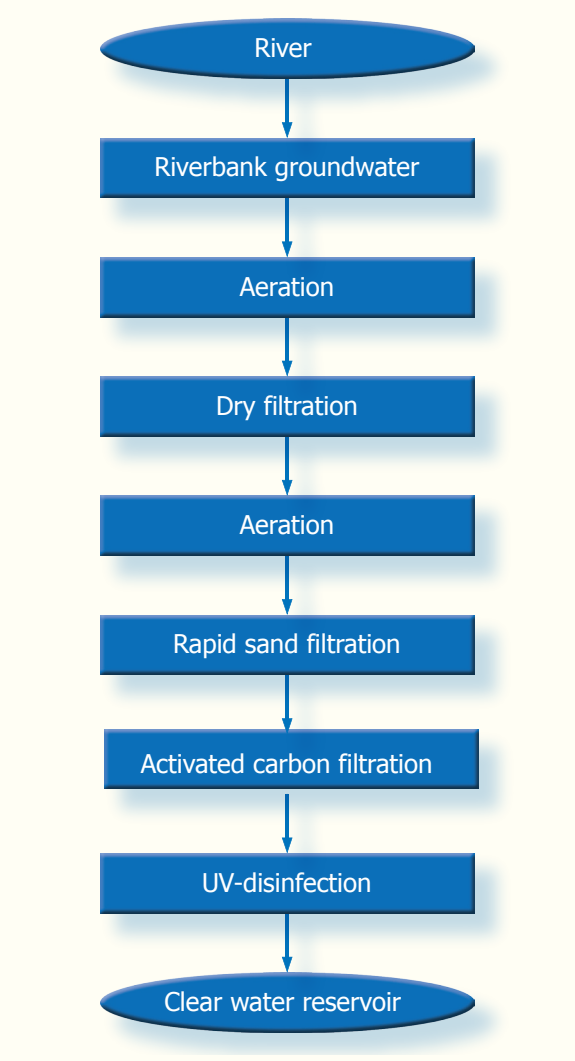



*Figure 11 - Treatment of riverbank groundwater*

*Figure 12 - Aeration over a dry filter in Zwijndrecht (Zuid Holland)*

ammonium, manganese and methane can be found. Furthermore, the hardness can be fairly high because of infiltration of river water. Due to high concentrations of ammonium, which are biologically transformed to nitrate, a lack of oxygen can occur in the treatment; therefore, an extra dry filtration stage is often included (Figure 12).

Activated carbon filtration is also used for the treatment of riverbank groundwater because of taste problems and problems with pesticides. A part of the water is surface water and, thus, it also contains substances associated with surface water.

For riverbank groundwater, UV-disinfection is often applied as the last disinfection stage, especially if activated carbon filtration is used in the treatment. In the activated carbon filters grow microorganisms due to the breakdown of organic material; these can subsequently end up in the water.
With UV-disinfection the microorganisms are killed, without the formation of disinfection by-products which are typical for chemical disinfection.

**Example**

As an example of riverbank groundwater, the Nieuw-Lekkerland pumping station of Hydron Zuid-Holland is described. Table 4 shows that the water contains a high concentration of ammonium and that there are pesticides present in the water.
Hence, the treatment scheme is as follows: aeration, dry filtration, aeration, submerged filtration, activated carbon filtration, and UV-disinfection.

Chloride can become a problem for riverbank groundwater treatment plants along the Rhine. In the treatment, chloride isn't removed. When the concentration in the Rhine is too high, then the standard for chloride may be exceeded.

Oxygen increases because of the aeration steps. Manganese and iron decrease because of the combination of aeration and filtration. Ammonium decreases because of transformation in the dry and submerged filters. Because of that the nitrate content increases. Because of activated carbon filtration, the Bentazon content decreases.

There are no E-coli in the raw water because the raw water passes through the soil. During UV-disinfection, possible organisms are killed that grow in the activated carbon filter.

*Table 4 - Quality data of raw and treated water at Nieuw-Lekkerland pumping station (Zuid-Holland)*

| Parameter | Unit | Raw water | Clear water |
|---|---|---|---|
| Temperature | °C | 12 | 12 |
| pH | - | 7.3 | 7.4 |
| EC | mS/m | 78.4 | 77 |
| SI | - | -0.2 | -0.1 |
| Turbidity | FTU | - | < 0.1 |
| $Na^+$ | mg/l | 69 | 70 |
| $K^+$ | mg/l | 4 | 4 |
| $Ca^{2+}$ | mg/l | 84 | 84 |
| $Mg^{2+}$ | mg/l | 12 | 12 |
| $Cl^-$ | mg/l | 128 | 135 |
| $HCO_3^-$ | mg/l | 223 | 187 |
| $SO_4^{2-}$ | mg/l | 55 | 59 |
| $NO_3^-$ | mg/l | < 0.1 | 2.3 |
| $O_2$ | mg/l | 0.8 | 5.7 |
| $CH_4$ | mg/l | 1 | < 0.05 |
| $CO_2$ | mg/l | 20 | 14 |
| $Fe^{2+}$ | mg/l | 3.8 | 0.02 |
| $Mn^{2+}$ | mg/l | 0.9 | < 0.01 |
| $NH_4^+$ | mg/l | 3 | < 0.03 |
| DOC | mg/l | 3 | 2.5 |
| E-Coli | n/100 ml | 0 | 0 |
| Bentazon | µg/l | 0.32 | < 0.05 |
| Chloroform | µg/l | - | - |
| Bromate | µg/l | - | - |

### *3.3 Riverbank filtrate*

The treatment of riverbank filtrate doesn't show many differences from the treatment of riverbank groundwater. Only in this case, the share of surface water is larger, which makes activated carbon filtration and post-UV-disinfection more important.
Dosing with ozone is applied in a number of cases for riverbank filtrate, oxidizing the micropollutants.
Dosing with ozone is, in this case, not necessary for disinfection. When the water is passing through the soil, all (harmful) bacteria are removed, even with a relatively short residence time.

For soil passage the filtration of microorganisms is the most important removal mechanism. For this, reference is made to the good microbiological water quality, obtained with slow sand filtration in surface water treatment. In a slow sand filter a good disinfection is obtained with a residence time of only 0.5 - 1.0 days.

## 4. Surface water with direct treatment

From a global point of view, the direct treatment of surface water is the most applied method for drinking water production. This is mainly because large cities have developed along river banks, making surface water directly available.
In order to be suitable for drinking water, suspended solids must be removed together with pathogenic bacteria. Over the years the removal of micropollutants has become necessary as well, together with the construction of storage basins, to use when the concentration of micropollutants is too high. Micropollutants often originate from human activities upstream.

The next section will describe the historical development of the direct treatment of surface water. Then, the contemporary treatment schemes will be described, followed by a description of future developments. Finally, the individual treatment processes in those schemes will be explained.

### *4.1 Historic development*

Throughout the ages, because of the increasing quantitative demand (due to population growth and consumption growth) and the increasing qualitative demand (because of worse sources and more stringent quality legislation), direct treatment methods for drinking water production have changed drastically.

Traditionally, direct treatment was performed by clarification in large sedimentation basins and subsequent slow sand filtration. Characteristic of this procedure was the enormous spatial demand and the labor intensive operation (manual removing of the "Schmutzdecke" from the slow sand filter).

By adding rapid filtration, the load on the slow sand filtration was decreased, making an increased production capacity with traditional means possible. To guarantee the bacteriological quality of the drinking water, a safety chlorination was applied as a final step in the treatment process. This caused a small amount of chlorine to be present in the water at the customers' taps.

In time production needed to be increased further. This caused the rapid filtration system to be heavily loaded, resulting in run times that were too short between backwashing. The problem was solved by adding a flocculant before sedimentation, thus increasing the effectiveness of the sedimentation.

When production demands increased further, the surface area of the slow sand filtration installation became the bottleneck. The slow sand filtration not only removed suspended solids, but removed (pathogenic) bacteria as well. Slow sand filtration was increasingly replaced by chemical disinfection (e.g., break-point chlorination). Break-point chlorination oxidizes ammonium ($NH_4^+$) to nitrogen ($N_2$) as well.

Increased river contamination necessitated the construction of reservoirs to be able to stop the direct intake of river water. Additionally, micropollutants needed to be removed by activated carbon (i.e., dosing of powdered activated carbon, PAC). This traditional treatment process (Figure 13) is still widely applied around the world.

The reservoirs were shallow basins at first. In these shallow reservoirs however, a considerable algal population can develop during spring and summer.
The first step in the treatment process is the application of microstrainers because of this algal bloom. Algae are quite difficult to remove by way of sedimentation, which is, in fact, only possible when using very high doses of flocculants. Additionally, algae can cause taste and odor problems.
Since the 1970s mainly deep reservoirs have been used. In these deep reservoirs, algae growth can be quite well controlled, making the microstrainers obsolete. Using deep reservoirs with a very long residence time will yield a considerable amount of self-purification in the basins as well.

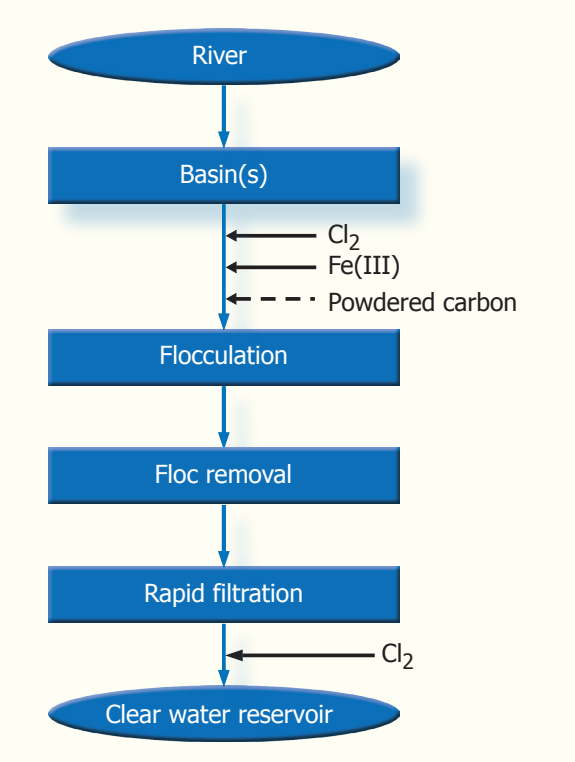



*Figure 13 - Tratitional treatment scheme for direct treatment of surface water*

Water from such reservoirs is typified by a low concentration of suspended solids (<5 mg/l), few algae, and a low ammonium concentration. With this water quality, sedimentation is sometimes unnecessary, and a very low dose of flocculant followed by rapid sand filtration can be used.

## *4.2 Contemporary treatment*

**Problems with traditional treatment**
Contemporary treatment originated from the chlorination issue and the increased river water pollution. In 1974 J. Rook, of the Rotterdam Water Company, discovered harmful by-products from the chlorination process (disinfectant by-products). These are mainly trihalomethanes (THMs), from which chloroform ($CHCl_3$) is produced at the highest level. THMs are created by the reaction of chlorine with humic acids present in the water, and are harmful to human health. The Dutch Decree on Water Supply sets a standard of 25 µg/l (sum) for THMs. Chlorination may lead to exceedence of this standard; but without sufficient chlorination, the disinfection would be inadequate, a worse condition from the point of view of public health. This caused the Dutch Decree to temporarily allow a THM value to 100 µg/l (until January 1, 2006).

In 1987 the insecticide Bentazon was found in Amsterdam's drinking water. Like many micropollutants, this insecticide proved to be insufficiently removed in the treatment process, even in the case of artificial infiltration into sand dunes. Other insecticides, like Atrazin and Diuron, have also been shown to pass through a traditional treatment process. Because activated carbon filtration does remove these pollutants sufficiently, it has become a typical element in any contemporary treatment process.

In 1993, a severe Cryptosporidium outbreak occurred in Milwaukee, Wisconsin (USA), resulting in 400,000 ill people and 100 deaths. Chlorination did not prove to be a sufficient barrier to cysts like Cryptosporidium and Giardia. Using higher doses and longer contact times will produce

trihalomethanes (THMs). Alternatives to this are stronger disinfectants like chlorine dioxide ($ClO_2$) or ozone ($O_3$).
Chlorine dioxide also produces by-products, but less than when using chlorine or hypochlorite. Disinfection using ozone can produce bromate, which is also harmful to public health. However, because sufficient disinfection is essential for the drinking water supply, the Dutch Decree on Water Supply allows an increase in the maximum bromate concentration from 1.0 µg/l to 5.0 µg/l when using ozone disinfection.

**Characteristics of temporary direct treatment**
Characteristics of the current treatment of surface water for production of drinking water are:

- storage reservoirs with a retention time of 1 - 3 months, making an intake stop possible in case of severe river contamination, and with a depth of over 20 meters to control algae growth
- process reservoirs with a retention time of about 1 month and a depth of over 20 meters, leading to a significant self-purification (sedimentation of suspended solids, ammonium oxidation) while still keeping algae growth under control
- removal of suspended solids by coagulation (adding flocculants), flocculation and floc removal by filtration, possibly preceded by sedimentation or flotation
- primary disinfection using a minimum amount of chlorine or ozone
- removal of micropollutants by activated carbon filtration
- secondary disinfection using a minimum amount of chlorine or chlorine dioxide

**Example of contemporary direct treatment (chlorine and activated carbon filtration)**
An example of current direct treatment can be found at the Berenplaat production plant (Figures 14 and 15).
At this site drinking water is produced from Meuse water, which has first been stored in the Biesbosch storage reservoirs. At the Berenplaat plant, microstrainers form the first step in the treatment scheme. This process was selected because, previously, the water, stored in a shallow basin, led to algae growth. In the current scheme the microstrainers could have been left out, but, actually, they have been left in service.

To disinfect the water, hypochlorite is added (about 1 mg/l as $Cl_2$). This needs to contact the water for half an hour, which is done in a tank with a canal labyrinth. After this phase a flocculant is added (ca. 5 mg/l $Fe^{3+}$ in the form of $FeCl_3$) for the coagulation of suspended solids, and then lime is added to correct the pH value of the water (ca. 6 mg/l in the form of CaO). When necessary, a flocculant aid (ca. 1 mg/l in the form of Wispro in winter) and powdered activated carbon (ca. 7.5 mg/l in case of severe pollution) are added. The added chemicals are mixed with the water using mechanical stirrers for rapid mixing.

The adding of $FeCl_3$ is for removing suspended solids that remain in the water after the storage

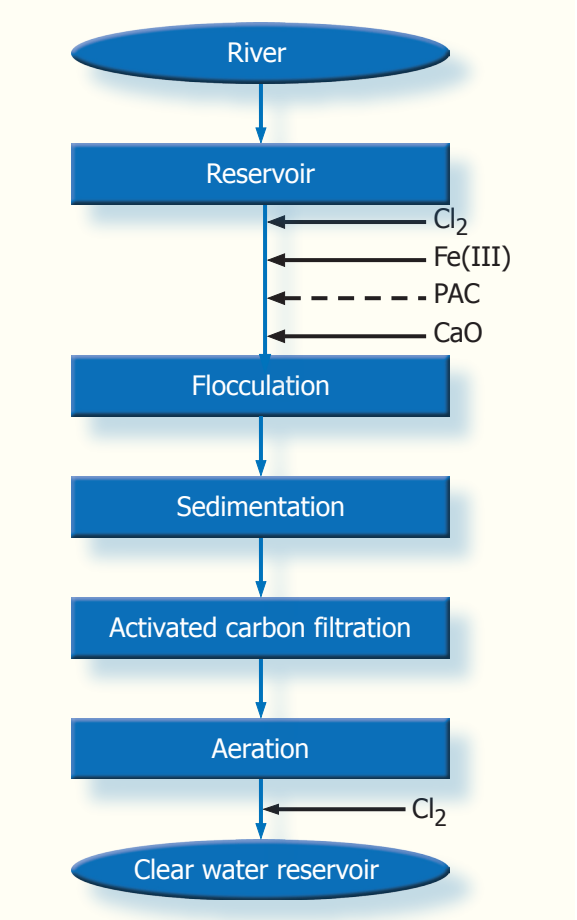



*Figure 14 - Contemporary direct treatment of surface water (Berenplaat, before 2006)*

reservoirs. The $Fe^{3+}$ together with the lime $OH^-$ form small $Fe(OH)_3$ flocs around the particles. Mechanical stirrers cause turbulence in the water, and the flocs collide and grow (flocculation). A flocculant aid can accelerate this process.

Because the flocs are heavier than water, they can be removed by sedimentation. This is done in the floc-blanket clarifier (floc removal). The total retention time in the floc-blanket clarifier is about one hour at a sedimentation rate of not more than 4.8 m/h. The settled flocs (sludge) are drained into a very large sedimentation basin, where they accumulate at the bottom.
To remove the remaining turbidity, taste, odor, and micropollutants, the water is treated using activated carbon filters. The filters consist of a layer of granular activated carbon at a height of 1.1 meters, applied over a supportive gravel layer. The filtration rate is no more than 9.4 m/h, equivalent to an approximate retention time (empty bed contact time) of 7 minutes minimum.

Because the filters will slowly clog, they need to be backwashed with clean water in an upward direction every few days. Because carbon activity decreases over time, the carbon must be reactivated every 1 - 1.5 years.

Cascades (five steps with a total height of about 2 meters) bring oxygen into the water; before that water is pumped into the clear water reservoirs. Aeration is included in the treatment process in order to add oxygen which could be low in the raw water because of biological processes. Chlorine dosing causes the biological activity during the treatment process to be minimal.
Hypochlorite is added before the clear water reservoirs (ca. 0.5 mg/l as $Cl_2$) to prevent regrowth during transportation.

*Figure 15 - Drinking water production at the Berenplaat production plant (Zuid-Holland)*

*Table 5 - Quality data of the raw and clear water at the Berenplaat drinking water production plant (Zuid-Holland)*

| Parameter | Unit | Raw water | Clear water |
|---|---|---|---|
| Temperature | °C | 11.9 | 11.9 |
| pH | - | 9 | 8.1 |
| EC | mS/m | 51 | 54 |
| SI | - | 0.9 | 0.1 |
| Turbidity | FTU | 2 | 0.1 |
| $Na^+$ | mg/l | 46 | 49 |
| $K^+$ | mg/l | 6 | 6 |
| $Ca^{2+}$ | mg/l | 51 | 54 |
| $Mg^{2+}$ | mg/l | 8 | 8 |
| $Cl^-$ | mg/l | 72 | 74 |
| $HCO_3^-$ | mg/l | 87 | 95 |
| $SO_4^{2-}$ | mg/l | 64 | 83 |
| $NO_3^-$ | mg/l | 3 | 3 |
| $O_2$ | mg/l | 11.1 | 10.8 |
| $CH_4$ | mg/l | - | - |
| $CO_2$ | mg/l | 0.3 | 1.3 |
| $Fe^{2+}$ | mg/l | - | - |
| $Mn^{2+}$ | mg/l | - | - |
| $NH_4^+$ | mg/l | - | - |
| DOC | mg/l | 3.6 | 2.6 |
| E-Coli | n/100 ml | 100 | 0 |
| Bentazon | µg/l | 0.2 | < 0.1 |
| Chloroform | µg/l | 0 | 38 |
| Bromate | µg/l | < 2 | 2.0 |

The data shown in Table 5 indicate the water quality of the untreated and treated water. The raw water has a high pH value, caused by sodium hydroxide softening in the Biesbosch reservoirs. By forming $Fe(OH)_3$ the pH value is reduced, and by adding lime it is raised again to the desired level. The suspended solids are mainly removed in the floc-blanket clarifiers and during (activated carbon) filtration. Chlorination causes an increased chloroform concentration and a reduced E.coli number.

**Example of contemporary direct treatment (ozone with activated carbon filtration)**

Another example of contemporary treatment is found at the Kralingen production plant (Figures 16 and 17). Here, drinking water is also produced from Meuse water from the Biesbosch reservoirs. At the Kralingen plant a flocculant is added first (ca. 4 mg/l $Fe^{3+}$ in the form of $FeCl_3$), before the water goes through a static mixer. This causes small $Fe(OH)_3$ flocs to form and to include pollutants from the water. If necessary, another flocculant aid (ca. 1 mg/l in the form of Wispro, during winter)

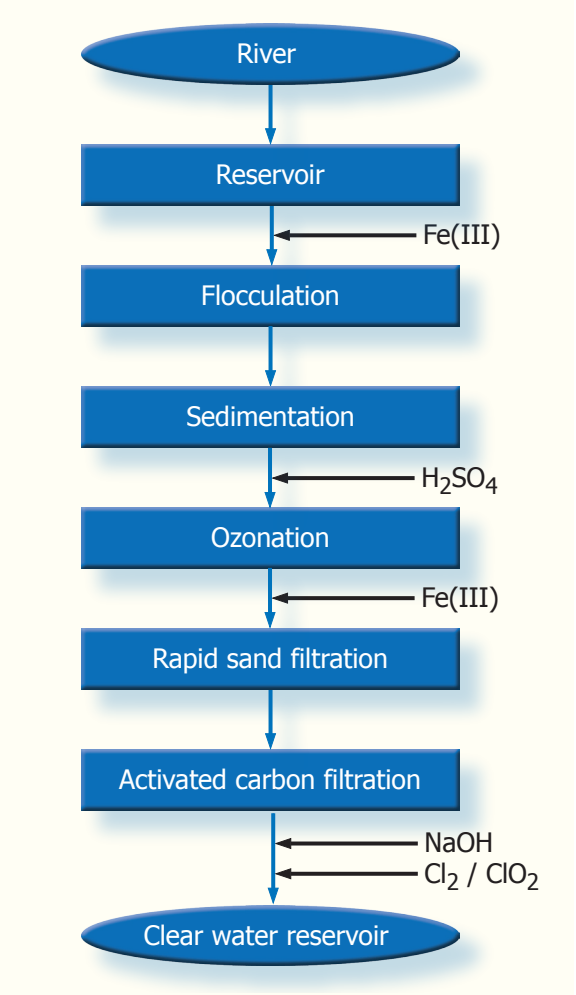



*Figure 16 - Contemporary direct treatment of surface water with ozonization*

is added. In four serial flocculation compartments having a total retention time of at least 20 minutes, slowly rotating mixers cause the flocs to grow. The mixing decreases in intensity in each consecutive compartment in order to prevent flocs from being destroyed. The flocs are removed in a lamella separator where they settle between ascending plates. This arrangement creates an enormous settling surface in a relatively small area. Particles with a sedimentation rate of over 1.2 m/h are all separated in this installation. The settled flocs slide down over the plates into a sludge thickener, which is equipped with stirrers. The thickened sludge is pumped to sludge-drying beds for dewatering.

After the flocculation and floc removal, sulfuric acid is added first to lower the pH, because ozone is more effective at low pH values. Ozone is produced locally from liquid oxygen. In ozone

*Figure 17 - Kralingen drinking water production plant*

generators the oxygen is exposed to high electric voltages, thus creating ozone. By means of a diffuser, the ozone is injected into the water (ca. 1.2 - 2.0 mg/l in the form of $O_3$). The ozone spreads through the water in the form of fine dissolving bubbles, being active there during a contact period of 8 - 10 minutes. The ozone gas that is released at the water's surface is destroyed thermally. Ozone kills bacteria and viruses, destroys micropollutants, and improves the taste of the water.
To remove the remaining turbidity, the water is treated in a dual-layer sand filter. For an effective performance of this filter, first an extra flocculant is added (ca. 0.5 mg/l $Fe^{3+}$ in the form of $FeCl_3$).

The sand filters have a surface area of 9 by 4 m and consist of a sand layer of 0.7 m and an anthracite layer of 0.8 m. Below these layers there is a gravel support layer. The filtration rate is 20 m/h maximum. Because the filters clog, they are backwashed daily with air (max. 80 Nm/h) and water (max. 45 m/h) in an upward direction.

Subsequently, the filtered water is treated with activated carbon for an approximate contact period (empty bed contact time) of 10 minutes. The remaining micropollutants and the taste and odor are removed. Because the activated carbon activity decreases in time, it needs to be reactivated every 1 - 2 years. Also, every two to three weeks the filters need to be backwashed in order to remove suspended solids. After the activated carbon treatment, sodium hydroxide is added to correct the pH value.

To make sure that microbiological regrowth does not occur during distribution, hypochlorite is added (ca. 0.3 mg/l in the form of $Cl_2$).

Table 6 shows the water quality of both the raw and treated water. The raw water has a high pH value, caused by the softening with sodium hydroxide in the Biesbosch reservoirs. Due to formation of $Fe(OH)_3$ and sulfuric acid, the pH value is reduced, and by adding sodium hydroxide it is increased again to the normal value. The suspended solids are mainly removed in the lamella separators and the dual-layer filters. Adding ozone results in an increased bromate content and a reduced E.coli number.

*Table 6 - Quality data of the raw and clear water of the Kralingen production plant (Zuid-Holland)*

| Parameter | Unit | Raw water | Clear water |
|---|---|---|---|
| Temperature | °C | 11.9 | 12.1 |
| pH | - | 9 | 8.2 |
| EC | mS/m | 51 | 55 |
| SI | - | 0.9 | 0.1 |
| Turbidity | FTU | 2 | 0.05 |
| $Na^+$ | mg/l | 46 | 52 |
| $K^+$ | mg/l | 6 | 6 |
| $Ca^{2+}$ | mg/l | 51 | 51 |
| $Mg^{2+}$ | mg/l | 8 | 8 |
| $Cl^-$ | mg/l | 72 | 73 |
| $HCO_3^-$ | mg/l | 87 | 94 |
| $SO_4^{2-}$ | mg/l | 64 | 85 |
| $NO_3^-$ | mg/l | 3 | 3 |
| $O_2$ | mg/l | 11.1 | 10.2 |
| $CH_4$ | mg/l | - | - |
| $CO_2$ | mg/l | 0.3 | 0.9 |
| $Fe^{2+}$ | mg/l | - | - |
| $Mn^{2+}$ | mg/l | - | - |
| $NH_4^+$ | mg/l | - | - |
| DOC | mg/l | 3.6 | 1.9 |
| E-Coli | n/100 ml | 100 | 0 |
| Bentazon | µg/l | 0.2 | < 0.1 |
| Chloroform | µg/l | 0 | 1.8 |
| Bromate | µg/l | < 2.0 | 3.9 |

Mainly, the concentrations of DOC and Bentazon are reduced during the activated carbon filtration. With ozone, the increased retention time, and the greater biological activity, DOC removal is better at Kralingen than at the Berenplaat production plant. The low chlorine dosing results in a small increase in chloroform in the water.

### *4.3 Future treatment*

**Problems of contemporary treatment**

Contemporary treatment techniques still face some problems, such as the by-products (THMs and bromates) that are formed during disinfection and oxidation, due to the discovery of new emerging micropollutants, and the required prevention of Legionella.

The effective removal of Cryptosporidium and Giardia requires high dose ozone. The tightened regulations regarding bromate make this more difficult. Besides, new and difficult to remove polar micropollutants have been discovered. These compounds may require an oxidation process

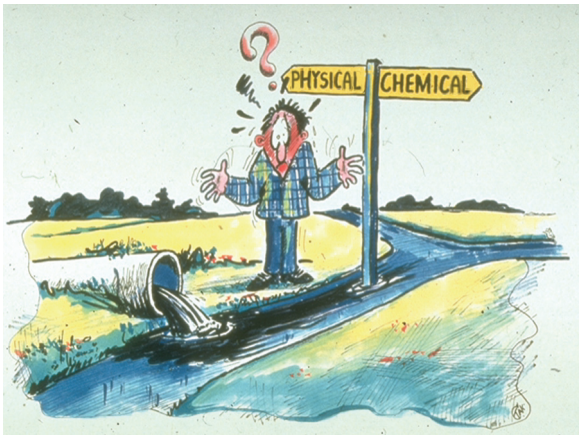



*Figure 18 - Future treatment of surface water: physical of chemical*

with high doses, which will again give rise to the formation of undesirable by-products.

Also, the increase of hormones in surface water (i.e., estrogen) and materials which act as endocrine disruptors and lead to hormonal deviations will be important in future drinking water production from surface water. Finally, the Legionella issue will require an improved water quality in order to reduce Legionella growth in the distribution network. This will require a further reduction in the amount of assimilable organic matter (AOC) in the water.

The above developments will necessitate a renewed orientation of the integral setup of treatment schemes for the direct production of drinking water from surface water. It may be that biological and physical processes will increasingly take over the role of the chemical processes for disinfection and oxidation (Figure 18).

**Biological processes**

In biological processes, many pollutants are assimilated by biomass and removed in this way. Also, biological processes will result in reduced amounts of organic matter (DOC, AOC, etc.). The treated water should be biologically stable, so that the biological activity in the distribution network will be low and residual disinfection is unnecessary.

The biological treatment processes that are currently considered for large-scale application are:

- biologically activated carbon filtration.
- slow sand filtration

*Figure 19 - UV disinfection (right) followed by activated carbon filtration (left) at Berenplaat production plant (Zuid-Holland)*

For Amsterdam's water supply some steps in this direction have been taken in recent years, and some aspects of it are currently operational. A further optimization of the contemporary treatment processes is being researched.

**Physical processes**

Physical processes currently being considered for large-scale applications are:

- UV disinfection (Figure 19)
- membrane filtration

By exposing the water to UV radiation, the DNA structure of organisms is destroyed, thereby stopping growth. It has proved to be very effective to combine UV disinfection with hydrogen peroxide as a strong oxidant. Both processes have not shown any harmful side-effects to date. An example of such a system is the Noord-Holland (Andijk) water supply (Figures 20 and 21).

With membrane filtration, the water is pressurized through a membrane. These membranes are available in several different pore sizes (Figure 22).

Ultra- and microfiltration mainly retain the coarser pollutants, like suspended solids, cysts and bacteria. Nanofiltration also retains divalent ions ($Ca^{2+}$, $SO_4^{2-}$ etc.), most larger organic compounds (humic acids), and most micropollutants. Here, cysts, bacteria and viruses are entirely filtered out. Reverse osmosis increases the filtration to monovalent ions and almost any micropollutant.

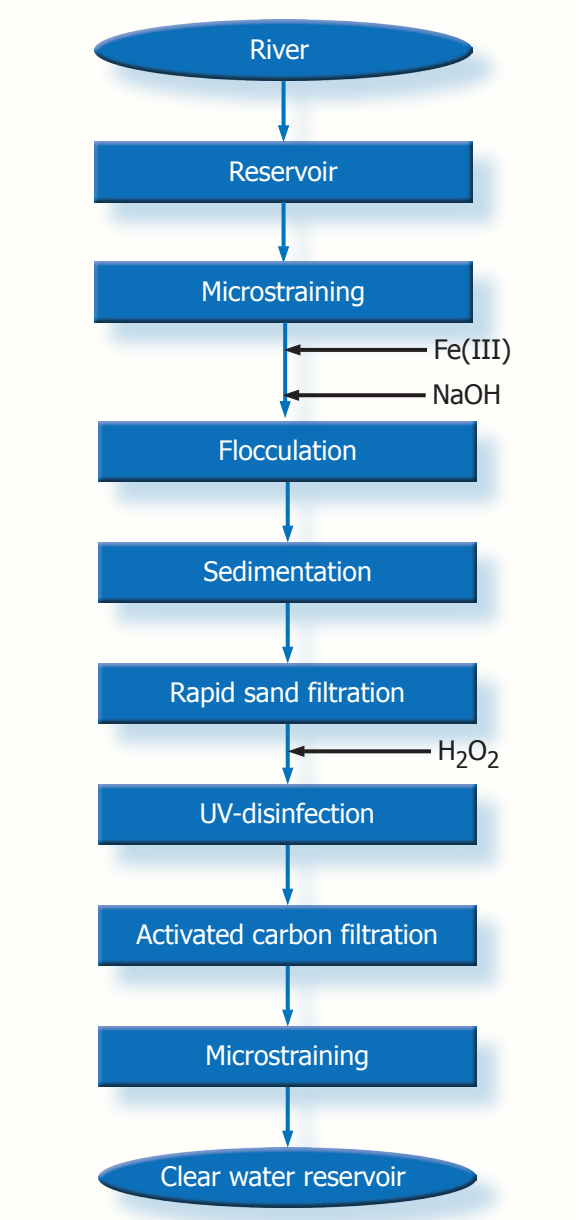



*Figure 20 - Direct treatment of surface water at Andijk production plant (2005)*

*Figure 21 - UV / $H_2O_2$ (Andijk)*

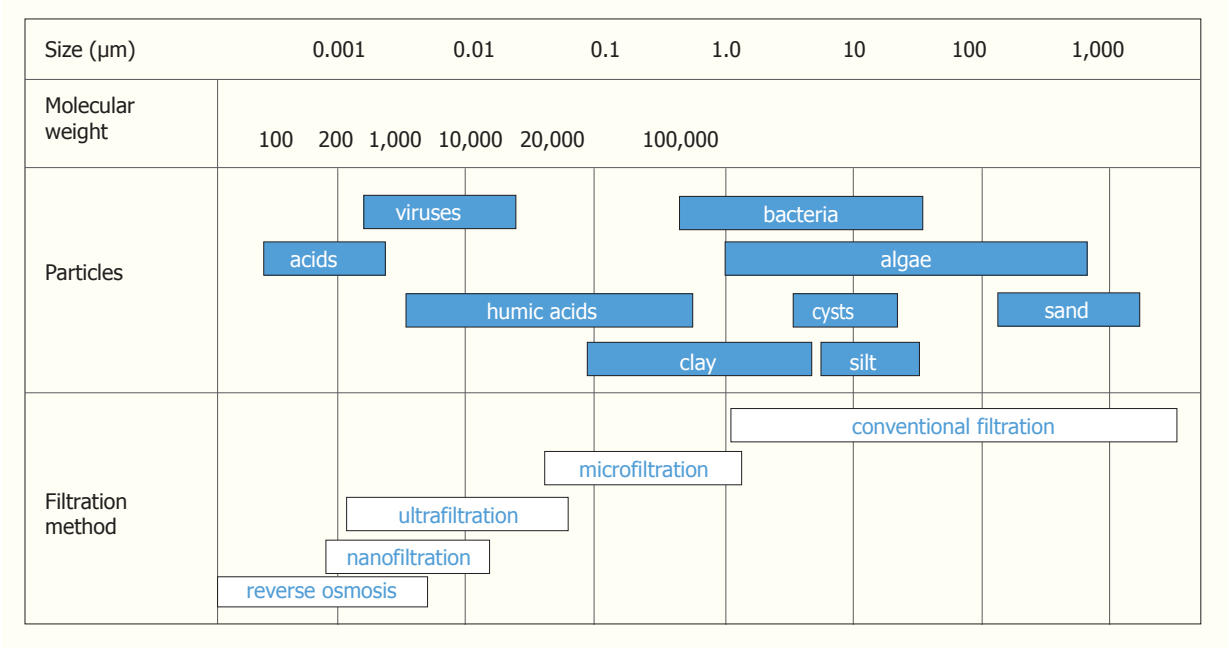



*Figure 22 - Application fields for membrane filtration*

There are some objections to the application of membrane filtration:

- risk of membrane defects and thus incomplete disinfection
- disposal of concentrate
- high costs of construction and operation

Recently, a treatment plant based on membrane filtration (ultrafiltration followed by reverse osmosis, Figures 23 and 24) was started up in Noord-Holland (Wijk aan Zee). The produced water is mixed with drinking water from a system of artificially infiltrated surface water.

## 5. Surface water with infiltration

### *5.1 Surface water with open infiltration*

The source of infiltration water is surface water. However, disadvantages of surface water are that the temperature and the salinity vary throughout the year, that it is contaminated with pathogenic microorganisms, and that, even after treatment, growth and settling of particles in the distribution network may occur.

By infiltrating the surface water into the ground its quality is improved. This means that the pathogenic microorganisms are degraded and that the

*Figure 23 - Membrane filtration plant in Heemskerek (Noord Holland)*

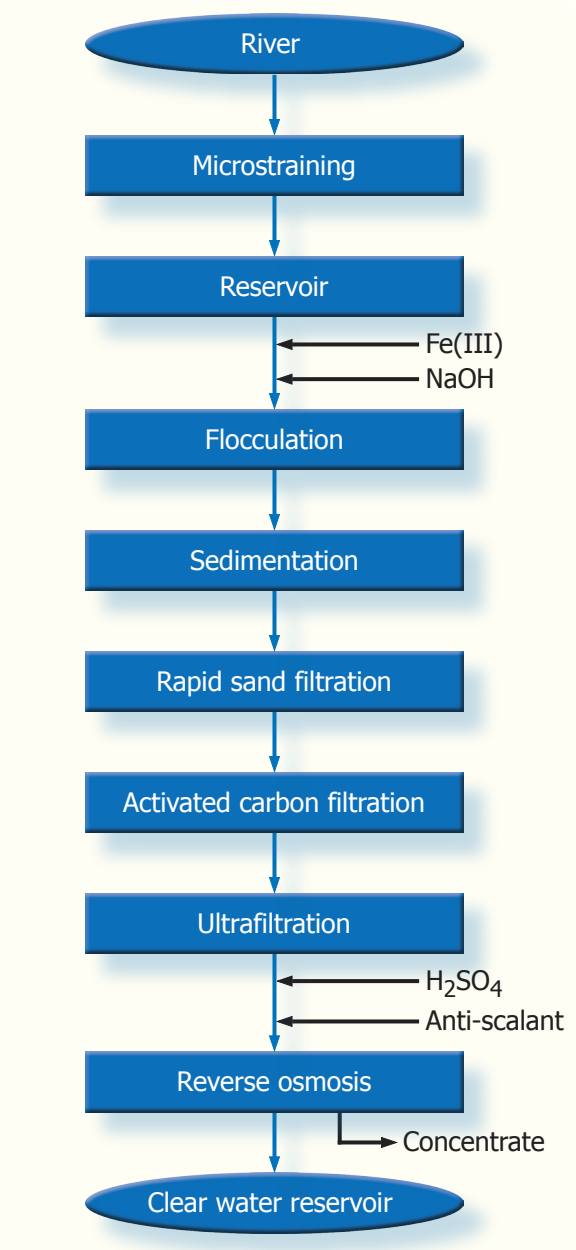



*Figure 24 - Direct treatment of surface water at Heemskerk production plant*

water is in a better biological and chemical state, causing no settling and no regrowth to occur. Also, the temperature changes are levelled. Both the temperature and the salinity will be more or less constant.

Water can be infiltrated into freatic groundwater. When the supply of treated surface water is obstructed somehow (e.g., accident, network repair), it is possible to continue the abstraction for some time. During this time the groundwater level

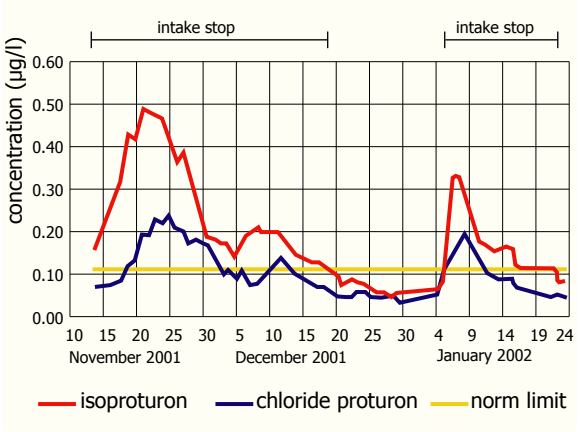



*Figure 25 - Pollution at the intake point of the WRK Nieuwegein*

decreases, but this is, to some extent, acceptable without damaging the natural biology.

Infiltration projects cover large areas that also need to be protected, because they mainly deal with large amounts of water. However, as the infiltration area is always developed as a natural area, it will have a high recreational value.

Figure 25 shows an example of an intake stop because of contamination of the surface water by insecticides. Because the contamination occurred in winter, an intake stop of some weeks could be taken without damaging the environment in the infiltration area.

### *5.2 Surface water with deep infiltration*

Expansion of natural infiltration into the dunes is not always possible because of environmental aspects. A water company using water from the dunes has, nevertheless, two possibilities to increase capacity.
First, the pre-treated surface water can be purified directly (see direct treatment), after which the water is mixed with the infiltrated water.

Second, the company may apply deep infiltration. Deep infiltration infiltrates the water into a confined aquifer. Because of the enclosing clay layers, there is almost no exchange with the freatic water above, so the infiltrated water does not influence the ecosystem.

aWhen using deep infiltration, care has to be taken during the preliminary treatment. The fewer particles that are present in the water, the smaller the chance that the infiltration wells will clog. Storage in deep infiltration is limited. When extracting water without supplying the necessary water for a long period, the chance of salt water intrusion exists.

Deep infiltration is not pursued only in the dunes. Also in other parts of the Netherlands, deep infiltration is used for the production of drinking water (Figure 26). The requirements are that the soil is sufficiently permeable and that there are confining layers in the underground.

### 5.3 Pre-treatment

To make the surface water suitable for transport and infiltration, suspended particles need to be removed first.

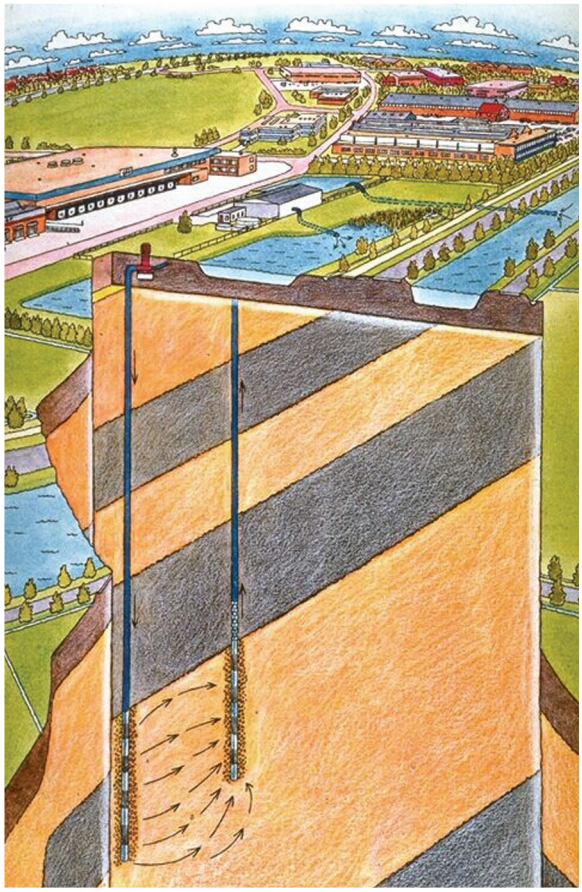


*Figure 26 - Deep infiltration at Someren (Noord Brabant)*

The standard pre-treatment of surface water consists of flocculation followed by floc removal and rapid filtration (Figure 27).

Flocculation is achieved by adding a flocculant, which removes the negative charge of the colloid particles, thereby making flocculation possible. These flocs can be removed by means of sedimentation in large ponds or in compact lamella separators, or by means of flotation.
Not all flocs are removed in sedimentation. Small flocs remain suspended and need to be removed by means of rapid filtration.
In the flocs, many other materials are removed like heavy metals (being positively charged and adsorbed to the flocs), and microorganisms. As a flocculant, mostly a trivalent metal salt, like iron chloride ($FeCl_3$), iron chloride sulfate, or aluminum sulfate, is used.

At the WRK Nieuwegein production plant (NV Watertransportmaatschappij Rijn-Kennemerland), flocculation is accomplished with iron chloride, followed by settling in a large sedimentation tank, and rapid filtration (Figure 27, Figure 28 and Table 7).

The quality of surface water varies throughout the year. For example, the turbidity at the Nieuwegein

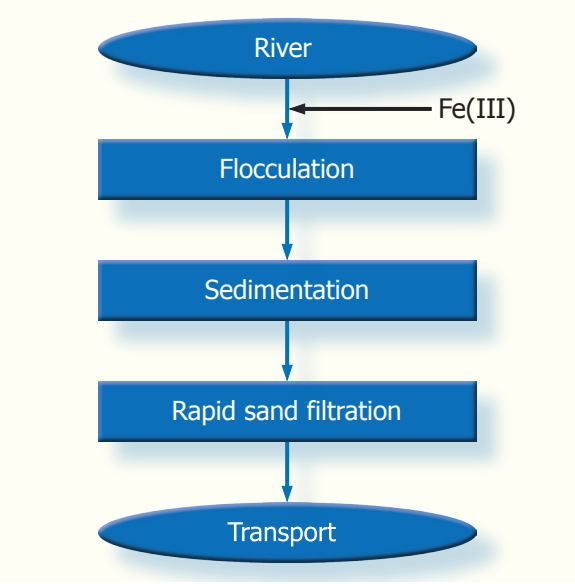



*Figure 27 - Preliminary treatment of surface water for infiltration*

*Table 7 - Quality data of the raw and treated water at Nieuwegein (Utrecht)*

| Parameter | Unit | Raw water | Clear water |
|---|---|---|---|
| Temperature | °C | 12.5 | 12.5 |
| pH | - | 8 | 7.8 |
| EC | mS/m | 80 | 80 |
| SI | - | 0.4 | 0.2 |
| Turbidity | FTU | 10.4 | 0.2 |
| $Na^+$ | mg/l | 80 | 81 |
| $K^+$ | mg/l | 6 | 6 |
| $Ca^{2+}$ | mg/l | 81 | 81 |
| $Mg^{2+}$ | mg/l | 11 | 11 |
| $Cl^-$ | mg/l | 149 | 155 |
| $HCO_3^-$ | mg/l | 157 | 156 |
| $SO_4^{2-}$ | mg/l | 66 | 67 |
| $NO_3^-$ | mg/l | 4 | 4 |
| $O_2$ | mg/l | 9.2 | 7.3 |
| $CH_4$ | mg/l | - | - |
| $CO_2$ | mg/l | 2.6 | 4.4 |
| $Fe^{2+}$ | mg/l | - | - |
| $Mn^{2+}$ | mg/l | - | - |
| $NH_4^+$ | mg/l | - | - |
| DOC | mg/l | 3.9 | 3 |
| E.Coli | n/100 ml | 5,000 | 50 |
| Bentazon | µg/l | 0.2 | 0.2 |
| Chloroform | µg/l | 0 | 0 |
| Bromate | µg/l | < 2.0 | < 2.0 |

site varies between 5.5 and 25.5 FTU and the temperature between 2 and 23°C. This influences the settling behavior, the filtration, and the biological processes.

During the treatment process the composition of the water changes. The quantity of suspended solids, the turbidity, the amount of heavy metals, like cadmium and nickel, and the colony count decrease.
By adding iron chloride, the chloride concentration of the water rises. On average 3 mg/l $Fe^{3+}$ is added,

*Figure 28 - Open sedimentation pond at Nieuwegein (Utrecht)*

implying an increase in the chloride concentration of 5.7 mg/l. The ferric ions form a compound together with the hydroxide ions, thus removing hydroxide ions and reducing the pH value.

The quality standards for infiltration water have been made stricter.
Originally, preliminary treatment was performed in order to prevent contamination of the transport pipelines and the clogging of the infiltration ponds. Nowadays, another demand is that no elements foreign to the infiltration environment will accumulate (like organic micropollutants). The standard pre-treatment process for infiltration water does not remove organic micropollutants like insecticides. This requires the preliminary treatment process to be expanded with activated carbon filtration (Figure 29).
At the WRK Andijk site, the preliminary treatment process consists of a reservoir, flocculation, floc removal (lamella settling), rapid filtration and activated carbon filtraton. Iron chloride sulfate

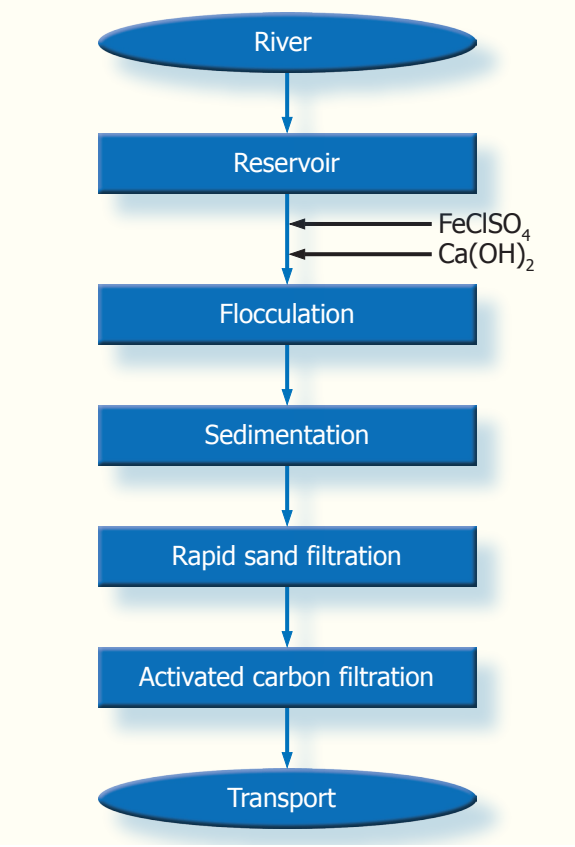



*Figure 29 - Treatment scheme of infiltration water where organic micropollutnats are removed*

($FeClSO_4$) is used as a flocculant. Growth of algea is common in the IJsselmeer. Therefore, the amount of organic matter (DOC) and the turbidity are high (Table 8). To reduce the algae the flocculant needs to be added in relatively large amounts, 20 mg/l $Fe^{3+}$ on average. This causes the pH value of the water to decrease. To increase the pH value, lime ($Ca(OH)_2$ is added.

Because WRK Andijk water is now used for both deep infiltration and membrane filtration, the requirements regarding the turbidity and the clogging capacity, expressed as MFI (membrane fouling index), have been increased. Those demands may be met by the current treatment process given that the process is well-controlled. At WRK Andijk no inexpensive iron chloride nor caustic soda is added, because the IJsselmeer contains large concentrations of chloride and sodium, which should not be further increased.

### 5.4 Infiltration

Pre-treatment is sited at the intake point of the surface water. This site is rather remote from the infiltration area. Infiltration, therefore, requires long transport pipes.

The pre-treated surface water infiltrates into the soil, which results in a quality improvement, including the levelling of concentrations.

A retention time of two months is deemed enough to make the water reliable, from a microbiological point of view.

The composition of the infiltrated water is different from the composition of the original dune water. Foreign water is infiltrated into the dunes. This will cause nutrients to enter the normally poor sand soil, thus changing the vegetation. In this way infiltration can affect natural areas.

After being abstracted out, the dune water is transported through pipes to the final treatment plant.

*Table 8 - Quality data of the raw and treated water at Andijk (Noord-Holland)*

| Parameter | Unit | Raw water | Clear water |
|---|---|---|---|
| Temperature | °C | 11 | 11.1 |
| pH | - | 7.8 | 7.8 |
| EC | mS/m | 95 | 80 |
| SI | - | 0.1 | 0.2 |
| Turbidity | FTU | 8 | 0.2 |
| $Na^+$ | mg/l | 113 | 81 |
| $K^+$ | mg/l | 9 | 6 |
| $Ca^{2+}$ | mg/l | 70 | 81 |
| $Mg^{2+}$ | mg/l | 14.5 | 11 |
| $Cl^-$ | mg/l | 150 | 160 |
| $HCO_3^-$ | mg/l | 138 | 115 |
| $SO_4^{2-}$ | mg/l | 37 | 67 |
| $NO_3^-$ | mg/l | 4 | 4 |
| $O_2$ | mg/l | 10.1 | 9.5 |
| $CH_4$ | mg/l | - | - |
| $CO_2$ | mg/l | 2.6 | 1.3 |
| $Fe^{2+}$ | mg/l | - | - |
| $Mn^{2+}$ | mg/l | - | - |
| $NH_4^+$ | mg/l | - | - |
| DOC | mg/l | 8 | 3.3 |
| E.Coli | n/100 ml | 5,000 | 50 |
| Bentazon | µg/l | 0.2 | < 0.1 |
| Chloroform | µg/l | 0 | 0 |
| Bromate | µg/l | < 2.0 | < 2.0 |

### 5.5 Final treatment of infiltrated water

After being abstracted, the water requires a final treatment.

The soil passage in the dunes removes micro-organisms and, at the same time, iron, manganese, and ammonium ions from the soil are dissolved. These ions need to be removed from the water. In case the water was not treated with activated carbon before infiltration, organic micropollutants may be present in the water. Those need to be removed as well.

The simplest procedure for the final treatment of infiltration water is similar to a groundwater treatment plant: aeration followed by rapid filtration (Figure 30).

As an example of the final treatment of infiltration water, the Bergen plant in Noord-Holland is considered.

Pre-treated water from the IJsselmeer (with activated carbon filtration) is infiltrated into the dunes. Abstracting is done by means of a closed system. The final treatment consists of aeration and rapid filtration.

Despite the fact that the hardness of the water is rather high (2.25 mmol/l), no softening process is applied.

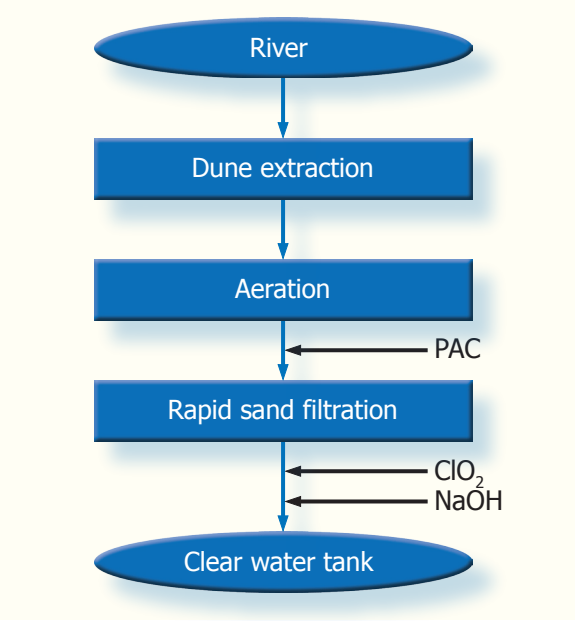



*Figure 30 - Quality data of the raw and treated water at Andijk (Noord-Holland)*

The abstracted water has a low turbidity, because it has been filtered by the dunes (Table 9).
During the retention time in the ground, the oxygen concentration is reduced. During the aeration phase the oxygen rate will rise again.

During aeration the iron and manganese ions are oxidized. To remove the iron hydroxide and manganese oxide, rapid filtration is used. The ammonium is transformed into nitrate in the rapid filter.

The concentrations of Bentazon and Diuron (pesticides) are low when abstracting water from the dunes. This is because of the activated carbon filtraton before infiltration. The concentrations will not be reduced further during the final treatment phase.

Generally, the applied treatment process is more extensive in practice. This is partly due to the treatment processes chosen in the past. Then, final treatment of infiltrated water consisted of slow sand filtration only. Later, other treatment techniques were added. The slow sand filters are still functional when the abstraction of water is done in an open system. In that case, the slow sand filters remove pathogenic microorganisms.

*Table 9 - Quality data of the raw and treated water at Bergen treatment plant*

| Parameter | Unit | Raw water | Clear water |
|---|---|---|---|
| Temperature | °C | 11.7 | 12 |
| pH | - | 7.6 | 7.7 |
| EC | mS/m | 86 | 86 |
| SI | - | 0.1 | 0.0 |
| Turbidity | FTU | - | 0.1 |
| $Na^+$ | mg/l | 78 | 78 |
| $K^+$ | mg/l | 7 | 7 |
| $Ca^{2+}$ | mg/l | 89 | 90 |
| $Mg^{2+}$ | mg/l | 13 | 12 |
| $Cl^-$ | mg/l | 147 | 149 |
| $HCO_3^-$ | mg/l | 192 | 198 |
| $SO_4^{2-}$ | mg/l | 92 | 93 |
| $NO_3^-$ | mg/l | 1 | 1 |
| $O_2$ | mg/l | 0.2 | 8 |
| $CH_4$ | mg/l | 0 | 0 |
| $CO_2$ | mg/l | 9 | 7 |
| $Fe^{2+}$ | mg/l | 0.6 | 0.03 |
| $Mn^{2+}$ | mg/l | 0.1 | < 0.01 |
| $NH_4^+$ | mg/l | 0.31 | < 0.03 |
| DOC | mg/l | 3 | 3 |
| E.Coli | n/100 ml | 0 | 0 |
| Bentazon | µg/l | < 0.1 | < 0.1 |
| Chloroform | µg/l | 0 | 0 |
| Bromate | µg/l | < 2.0 | < 2.0 |

Infiltration water from the large rivers and from the IJsselmeer has a high hardness. Softening with pellet reactors is undertaken to reduce the hardness to about 1.5 mmol/l. These pellet reactors are used after aeration and before rapid filtration, in order to remove the carry-over.

When they have not been removed during preliminary treatment, organic micropollutants are removed by means of powdered carbon or activated carbon filtration. The addition of powdered carbon is done before weir aeration, because that process will sufficiently mix the powder with the water. When activated carbon filtration is used, it is done after rapid filtration.
To improve the removal of organic micropollutants, ozonation can be used. Ozonation oxidizes organic macro-molecules into smaller organic molecules, which can be removed more easily during activated carbon filtraton. A disadvantage of the combination of ozonation and activated carbon filtration is that there will be a high rate of biological activity in the activated carbon filters. This may cause microorganisms to be present

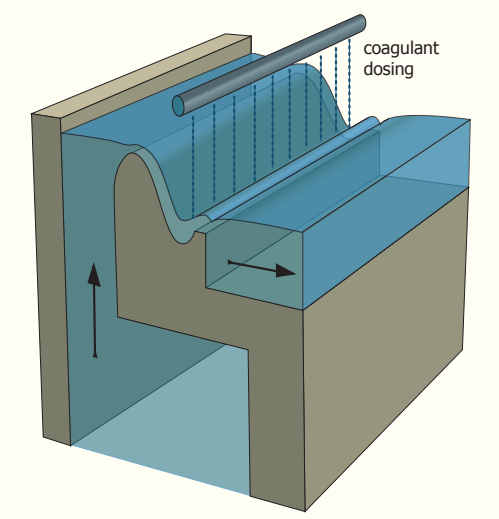
*Figure 31 - Infiltration area and post-treatment plant near Scheveningen*

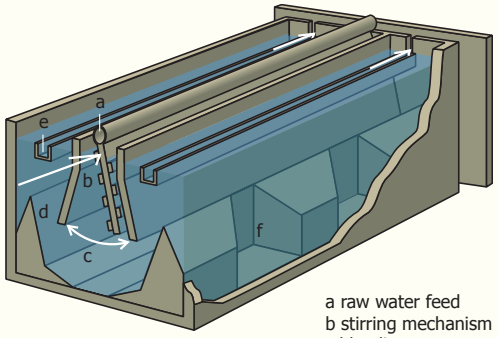
in the water, which will require disinfection after activated carbon filtration. This disinfection is done in the slow sand filters.

All in all this makes for quite an extensive treatment process, especially considering the fact that the water has been pre-treated before infiltration and transported over large distances. Therefore, the price of drinking water prepared from infiltration water is higher than that of water produced from groundwater.

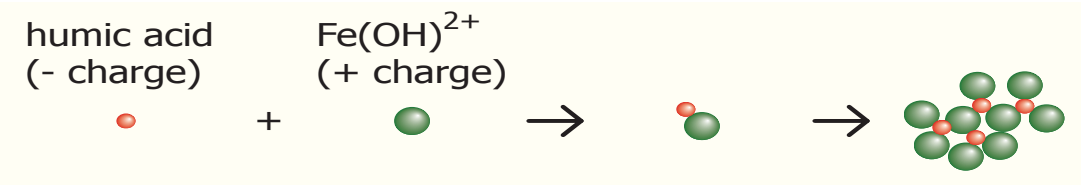
WATER TREATMENT

# Coagulation and flocculation




a raw water feed
b stirring mechanism
c blending space
d floc blanket
e clear water exit
f floc exit



## Framework

This module represents coagulation and flocculation.

## Contents

This module has the following contents:

1. Introduction
2. Coagulation
   2.1 Theory of coagulation
   2.2 Coagulation in practice
3. Flocculation
   3.1 Theory of floc formation
   3.2 Floc formation in practice

## 1 Introduction

In surface water different compounds are present that must be removed if drinking water is to be produced. The compounds can be subdivided into:
- suspended solids
- colloidal solids
- dissolved solids.

Suspended solids have a diameter larger than $10^{-6}$ m, colloidal solids between $10^{-9}$ and $10^{-6}$ m and dissolved solids smaller than $10^{-9}$ m.

Particles with a diameter larger than $10^{-5}$ m, and a specific density larger than 2000 kg/m$^3$ will settle in water. Smaller particles will also settle, but more slowly.
In Table 1 the settling time of particles with a density of 2650 kg/m$^3$ (e.g., sand) is given.
To be removed, particles that are smaller than $10^{-5}$ m must be made larger or heavier. The latter is impossible and, therefore, removal is only possible by increasing the particle size.
During the coagulation process, coagulants are added to the water to aid in floc formation. These flocs are precipitates in water, wherein small particles are incorporated.

To express the concentration of compounds in water, sum parameters are used. The most important sum parameters for surface water are “suspended solids” concentration (dry weight), “turbidity,” “natural organic matter” (expressed in TOC/DOC) and “color.”
“Suspended solids” concentration and turbidity

*Table 1 - Settling time of particles with a density of 2,650 kg/m$^3$*

| Diameter (m) | Types of particles | Settling time over 30 cm |
|---|---|---|
| $1.10^{-2}$ | gravel | 0.3 sec |
| $1.10^{-3}$ | coarse sand | 3 sec |
| $1.10^{-4}$ | fine sand | 38 sec |
| $1.10^{-5}$ | silt | 33 min |
| $1.10^{-6}$ | bacteria | 35 hours |
| $1.10^{-7}$ | clay | 230 days |
| $1.10^{-8}$ | colloids | 63 years |

*Figure 1 - Turbidity in surface water*

(Figure 1) are caused by colloidal particles (order of magnitude 0.1 - 10 µm). Colloidal particles are negatively charged and repulse each other.
In the tropics high concentrations of suspended solids can occur and rivers can be become “mud flows” (Figure 2).
Color (Figure 3) is caused by humic substances (order of magnitude 0.01 µm). The charge of humic substances (and thus the removal) is dependent upon the pH of the water.

In Table 2 the water quality data from the surface water of several rivers in the Netherlands and in tropical countries are given. The high values of organic matter and color in the Drentsche Aa are caused by peat-containing soils (with high organic matter content) that the river crosses.

## 2 Coagulation

The coagulation process is the dosing of a coagulant in water, resulting in the destabilization of

*Figure 3 - Color in water*

*Figure 2 - Rivers in the tropics sometimes **have high** suspended solids contents*

negatively charged particles.

## 2.1 Theory of coagulation

### Coagulants

To remove particles present in water, the particles must be incorporated into flocs. These flocs will be formed after dosing coagulant.
In the Netherlands iron chloride ($FeCl_3$) is frequently used as the coagulant. Alternatively, aluminum sulfate ($Al_2(SO_4)_3$) can be applied.

### Iron

Iron chloride is easy to dissolve in water; the solubility product ($K_{sp}$) is 27.9 $mol^4 \cdot l^{-4}$. Consequently, 162 g $FeCl_3$ can be dissolved in one liter of water, resulting in 55.8 mg/l $Fe^{3+}$ and 106.5 mg/l $Cl^-$.

In addition to other ions, the ions $Cl^-$, $SO_4^{2-}$, $Na^+$, $Ca^{2+}$, $H_3O^+$- and $OH^-$ are dissolved in water.
The $OH^-$ ions play an important role in coagulation. $Fe^{3+}$- and $OH^-$ ions precipitate, because the solubility product of iron hydroxide is low. Since $K_{sp\ Fe(OH)3} = 1 \cdot 10^{-38}$ $mol^4 \cdot l^{-4}$, only $7.8 \cdot 10^{-10}$ mol/l $Fe^{3+}$ and $2.34 \cdot 10^{-9}$ mol/l $OH^-$ ions can be present in water.
When the concentration of these ions is higher, they will precipitate into $Fe(OH)_3$-flocs.

When the pH of the surface water is known, the concentration of iron ($Fe^{3+}$) ions can be calculated using the solubility product of iron hydroxide and the ion product of water:

$$Fe^{3+} + 3 \cdot OH \rightarrow Fe(OH)_3 \qquad K_{sp} = 1 \cdot 10^{-38}$$

$$2 \cdot H_2O \rightleftarrows H_3O^+ + OH^- \qquad K_w = 1 \cdot 10^{-14}$$

Rewriting the water equilibrium results in the following equation:

$$K_w = 1 \cdot 10^{-14} = [H_3O^+] \cdot [OH^-] \Rightarrow [OH^-] = \frac{1 \cdot 10^{-14}}{[H_3O^+]}$$

Combining the equation mentioned above with the solubility product of iron hydroxide gives:

$$[Fe^{3+}] \cdot [OH^-]^3 = 1 \cdot 10^{-38} \Rightarrow$$

$$[Fe^{3+}] = 1 \cdot 10^{-38} \frac{[H_3O^+]^3}{(1 \cdot 10^{-14})^3} = 1 \cdot 10^4 \cdot [H_3O^+]^3$$

$$\log[Fe^{3+}] = \log(1 \cdot 10^4) + 3 \cdot \log[H_3O^+] = 4 - 3pH$$

In addition to iron hydroxide the following hydrolyses products of $Fe^{3+}$ are also formed:
$Fe(OH)^{2+}$, $Fe(OH)_2^+$, $Fe(OH)_4^-$.
In Table 3 the solubility constants of different reactions are given. From here, after some calculation, Figure 4 can be constructed.

*Table 2 - Water quality data of several rivers*

| River | Suspended solids (mg/l) | Turbidity (FTU) | Color (mg Pt/l) | DOC (mg/l) |
|---|---|---|---|---|
| Rhine | 9 - 53 | 5.5 - 22.5 | 9 - 17 | 3.1 - 6 |
| Meuse | 4 - 31 | 2.2 - 27 | 10 - 22 | 3.4 - 5.4 |
| Biesbosch reservoirs | 1.5 - 9 | 0.9 - 5.6 | 6 - 12 | 3.2 - 4.0 |
| IJsselmeer | 4 - 115 | 2.5 - 4.0 | 10 - 30 | 5 - 13.3 |
| Drentsche Aa | 2 - 20 | 3.4 - 39 | 10 - 100 | 4.8 - 14.9 |
| Tropical river | 10,000 | 5,000 | 1,000 | 500 |
| Drinking water | < 0.05 | < 0.1 | < 20 | 1 |

*Table 3 - Iron hydrolyses reactions*

| Iron hydrolyses reactions | | | $K_S$ |
|---|---|---|---|
| $Fe(OH)_3$ | ←→ | $Fe^{3+} + 3OH^-$ | $1 \cdot 10^{-38}$ |
| $Fe^{3+} + 2H_2O$ | ←→ | $Fe(OH)^{2+} + H_3O^+$ | $6.8 \cdot 10^{-3}$ |
| $Fe(OH)^{2+} + H_3O^+$ | ←→ | $Fe(OH)^{2+} + H_3O^+$ | $2.6 \cdot 10^{-5}$ |
| $2Fe^{3+} + 4H_2O$ | ←→ | $Fe_2(OH)_2^{4+} + 2H_3O^+$ | $1.4 \cdot 10^{-3}$ |
| $Fe(OH)_3 + OH^-$ | ←→ | $Fe(OH)_4^-$ | $1.0 \cdot 10^{-5}$ |

When the pH of water is known, the amount of hydrolysis product in a volume of water can be determined. With a pH of 4.6, $10^{-7}$ mol/m³ $Fe^{3+}$, $10^{-4}$ mol/m³ $FeOH^{2+}$ and $10^{-4}$ mol/m³ $FeOH_2^+$ are present.
The pH and the predominant hydrolysis product influence the predominant coagulation mechanisms.

As a result of the dosing iron chloride, $OH^-$ ions are removed and the pH will decrease.
The magnitude of the pH drop depends on the buffering capacity of the water. The higher the buffering capacity, the smaller the pH drop is. When the pH drop is too large, pH will be increased by dosing a base, such as caustic soda.

**Alumn**

Aluminum sulfate (alumn) is easy to dissolve in water and $Al^{3+}$ en $SO_4^{2-}$ ions will form. $Al(OH)_3$ has a low solubility product and precipitates.
By dissolving aluminum sulfate, different hydrolyses products are formed and, after calculating the solubility products, Figure 5 can be drawn.

**Coagulation mechanisms**

Destabilization of turbidity and color-causing

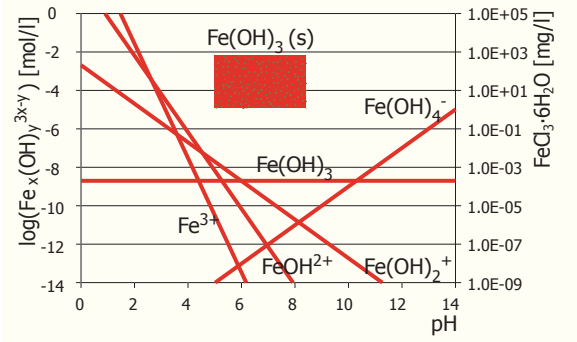



*Figure 4 - Forms of iron hydrolyses products*

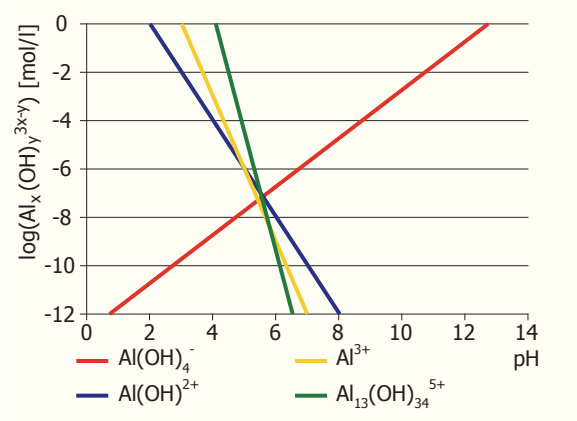



*Figure 5 - Forms of aluminum hydrolyses products*

substances can be induced by different mechanisms.
The following subdivisions can be made:
- electrostatic coagulation
- adsorptive coagulation
- precipitation coagulation.

**Electrostatic coagulation**

In electrostatic coagulation, positively charged ions approach the negatively charged colloids. In the diffusive layer around the colloid, the positively charged ions accumulate, destabilizing the colloid.

In Figure 6 the mechanism of electrostatic coagulation is represented.
In the left part of the figure the negatively charged colloids are shown. After dosing coagulant (right part of the figure), the exterior of the colloidal particle is destabilized and can collide with other particles forming removable flocs.

Electrostatic coagulation occurs after dosing 0.025 mol/m³ of trivalent ions. These concentrations are not found in surface water treatment and thus elec-

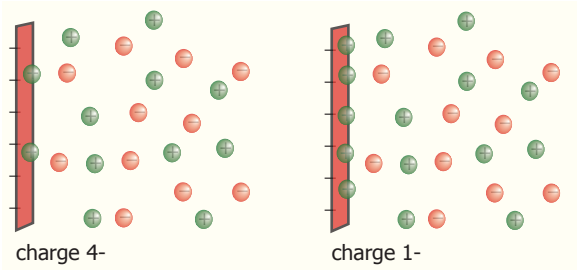



*Figure 6 - Mechanism of electrostatic coagulation*

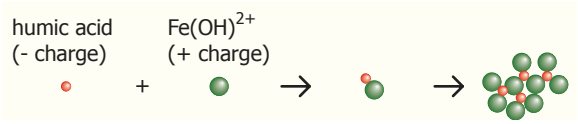



*Figure 7 - Mechanism of adsorptive coagulation*

trostatic coagulation does not play an important role in water treatment.

**Adsorptive coagulation**

In adsorptive coagulation, particles are adsorbed to the positively charged hydrolyses products $FeOH^{2+}$ and $FeOH_2^+$.
These products mainly occur at low pH (Figure 4). The optimal pH-range for adsorptive coagulation with iron salts is between 6 and 8; the optimal pH-range with aluminum salts is narrower and is about 7.

Characteristics of adsorptive coagulation are that dosing is proportional to the removal of organic matter and that restabilization can occur after an overdose of coagulant. After an overdose, the colloids will be positively charged and repulsion of the particles will take place.
Adsorptive coagulation is a rapid process. Within one second, positively charged hydrolyses products are formed and are adsorbed to the negatively charged particles.

**Precipitation coagulation**

In precipitation coagulation, or sweep coagulation, colloids are incorporated into neutral (iron) hydroxide flocs. This mechanism occurs mainly in waters with low suspended solids content (10 mg/l). In order to form hydroxide flocs, more coagulant must be dosed than is necessary for adsorptive coagulation.

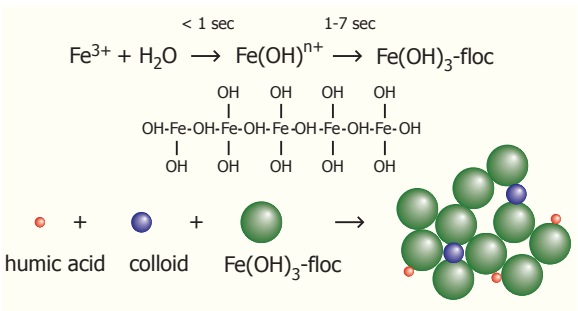



*Figure 8 - Mechanism of precipitation coagulation*

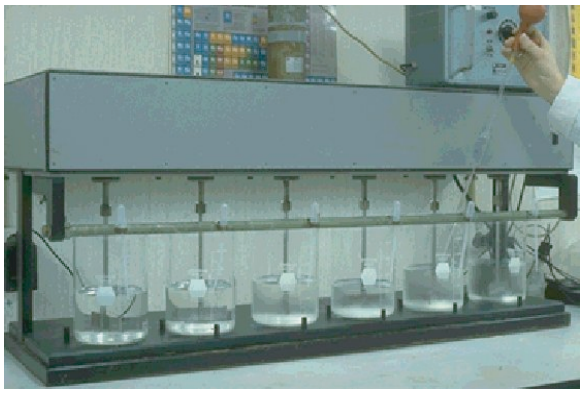


*Figure 9 - Jar-test apparatus*

## 2.2 Coagulation in practice

**Jar test**

The coagulation process can be researched by executing jar tests. In this test the coagulation and floc formation process is simulated.
The jar-test apparatus consists of 6 jars filled with water (Figure 9). To each jar a certain dose of coagulant is added. After rapid mixing, a slow stirring, and a settling phase, the water turbidity is measured.
By modifying the process conditions (dosage, pH, flocculation time, settling time, stirring energy for mixing and/or flocculation), the optimal conditions can be determined.

**Mechanisms**

The coagulation mechanisms discussed above occur in practice in parallel. This can be illustrated by discussing the results of several jar-test experiments.

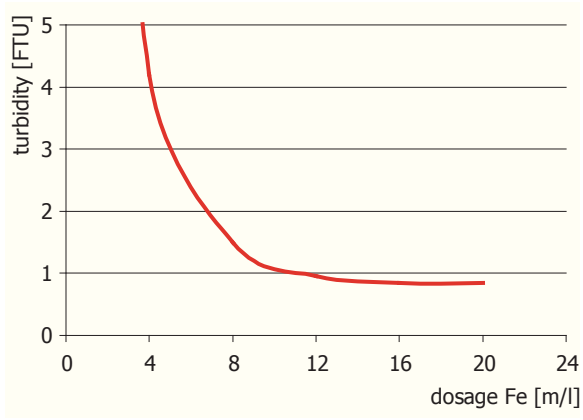



*Figure 10 - Results of jar-test experiment with varying coagulant dosing*

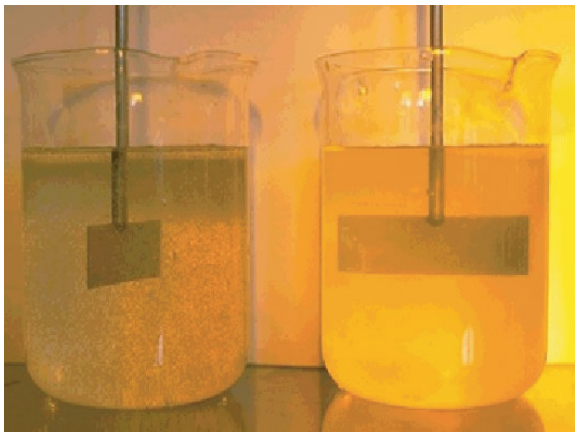


*Figure 11 - Influence of coagulant dose (left: high dose, right: low dose)*

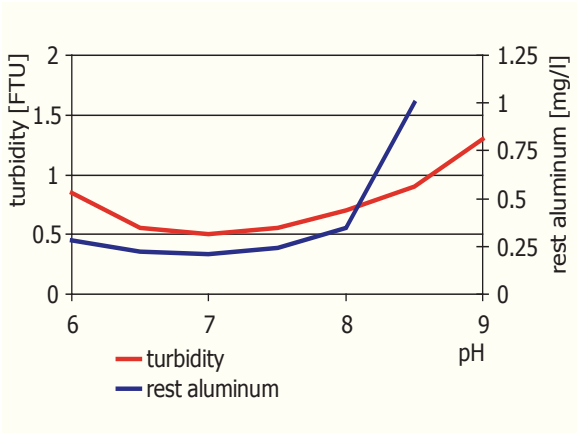



*Figure 13 - Results of jar-test experiment of "polder water" with varying pH*

In Figure 10 the results of a jar-test experiment of Biesbosch water is shown. Biesbosch water originates from the river Meuse and is collected in reservoirs. Due to the long retention times (about 6 months) in the reservoirs, the suspended solids concentration of Biesbosch water is low, about 5 mg/l.

It can be concluded from the figure that turbidity decreases with an increased coagulant dosing. The lowest turbidity is attained when about 12 mg/l iron chloride is dosed. With a higher dosage the turbidity does not increase and thus restabilization does not occur.

In Figure 12 a coagulant dose of 12 mg/l and a varying pH is represented. The turbidity increases with a decreasing pH (pH<7).

The predominant coagulation mechanism of Biesbosch water is precipitation coagulation.

In the province of Zeeland in the Netherlands, the drinking water company takes its water in from a polder as its source for the water treatment. Polder water has a high content of organic matter (like humic acids).

In Figure 13 the results are represented for jar-test experiments in which the coagulant dose varied with pH. At pH between 6 and 7 the lowest turbidity is found. At higher pH the turbidity is higher.

The prevailing mechanism is thus adsorptive coagulation.

An evident difference between the adsorptive and precipitation mechanisms is encountered during the coagulation of water from the Rhine. The water transport company Rijn-Kennemerland abstracts raw water from the river at Nieuwegein (WRK I-II) and from the IJsselmeer (lake) at Andijk (WRK III).

Although both water sources originate in the river Rhine, the coagulation mechanisms differ strongly

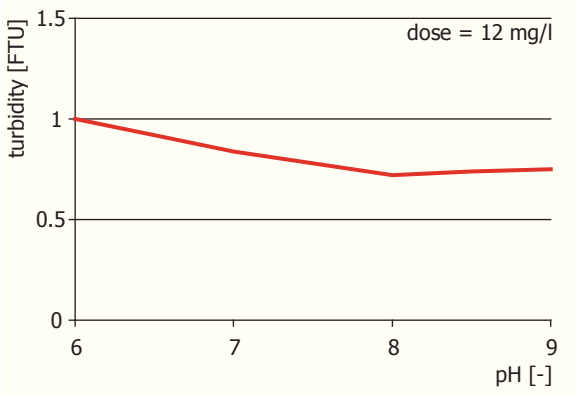



*Figure 12 - Results of jar-test experiment with varying pH*

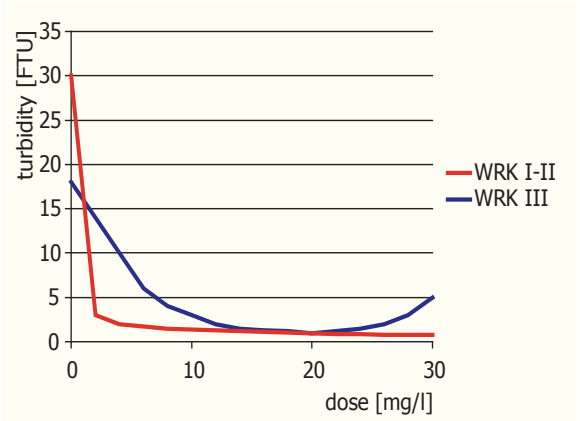



*Figure 14 - Coagulation of Rhine water*

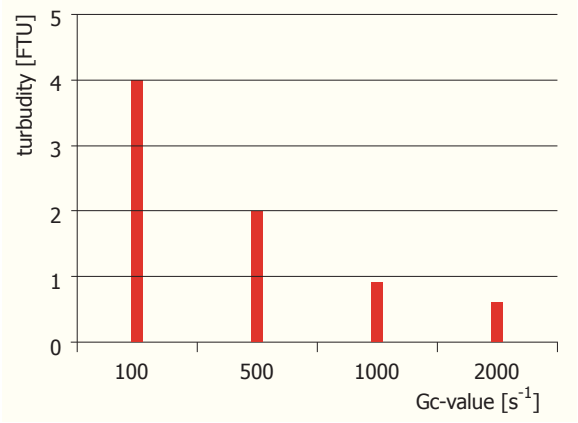



*Figure 15 - Rest turbidity at different $G_c$-values*

(Figure 14). The river water has a higher turbidity than the lake water; the lake water has a higher humic acid content than the river water due to algae bloom in summer.
During coagulation of lake water, restabilization can occur and the prevailing mechanism is adsorptive coagulation.
Restabilization is not detected in the coagulating river water and, therefore, the prevailing mechanism is precipitation coagulation.

**Mixing**

Rapid mixing after coagulant dosing is an important design parameter. The coagulant must be uniformly mixed with the raw water. In case mixing is poor, local under- and overdosing occurs, resulting in poor performance of the process.
The parameter expressing mixing intensity is called the velocity gradient or G-value.

The velocity gradient is defined as follows:

$$G_c = \sqrt{\frac{P}{\mu \cdot V}}$$

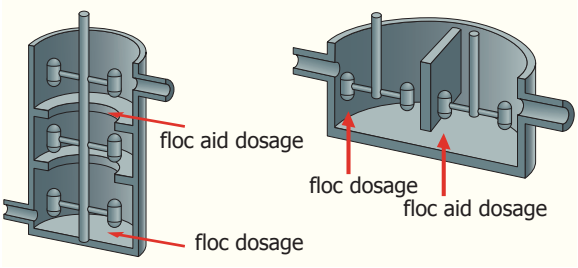



*Figure 16 - Mechanical mixers*

in which:

| | | | |
|---|---|---|---|
| $G_c$ | = | velocity gradient for rapid mixing | [$s^{-1}$] |
| P | = | dissipated power | [W] |
| $\mu$ | = | viscosity | [$N \cdot s/m^2$] |
| V | = | volume of mixing tank | [$m^3$] |

The influence of the velocity gradient can be determined by jar-test experiments (Figure 15).
When the velocity gradient is low (less intensive mixing), the residual turbidity will be higher than in situations where the velocity gradient is high.
In practice, the recommended G-value for rapid mixing is 1500 $s^{-1}$, at a minimum.

Two different mixing systems can be applied:
- mechanical mixing
- static mixing

In the first system mechanical mixers dissipate the power in the raw water, while in the second system gravity forces cause the mixing effect. Here, the dissipated power is a consequence of the head loss over the mixing tank:

$$P = \rho \cdot g \cdot Q \cdot \Delta H$$

in which:

| | | | |
|---|---|---|---|
| $\rho$ | = | density of water | [$kg/m^3$] |
| g | = | gravity constant | [$m/s^2$] |

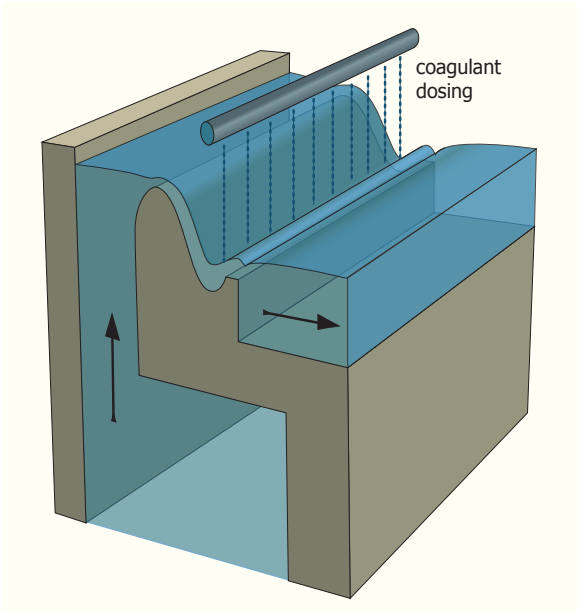



*Figure 17 - Cascade mixer*

Q = flow [$m^3/s$]
$\Delta H$ = head loss over mixing tank [m]

The equation for the velocity gradient for static mixers can be written as:

$$G_c = \sqrt{\frac{\rho \cdot g \cdot \Delta H}{\mu \cdot \tau_c}}$$

in which:
$\tau_c$ = residence time in the mixing zone [s]

The most frequently applied static mixer is the cascade. Water falls over a weir into a receiving body. In the turbulent space that is caused by the falling water, coagulant is dosed.

# 3 Flocculation

## *3.1 Theory of floc formation*

After coagulation and the resulting destabilization of particles, the particles must collide. The collision of particles can take place under natural circumstances (perikinetic floc formation) or by dissipation of mixing energy (orthokinetic floc formation).

**Perikinetic floc formation**
During perikinetic floc formation, particles collide as a result of Brownian motion. Von Smoluchowski described the decrease in the total number of spherical particles as a function of time with the following equation:

$$-\frac{dn}{dt} = \alpha \cdot \frac{4 \cdot k \cdot T}{3 \cdot \mu} \cdot n^2$$

in which:
n = total number of particles per unit water volume [$m^{-3}$]
α = collision efficiency [-]
K = Boltzmann constant [$J \cdot K^{-1}$]
T = absolute temperature [K]
μ = dynamic water viscosity [$N \cdot s \cdot m^2$]

Not every collision will result in attachment and therefore the collision efficiency is incorporated into the equation.

From experiments it can be concluded that perikinetic floc formation is a fast process but results in poor settling characteristics of the formed flocs.

**Orthokinetic floc formation**
By mixing, the collision frequency of the particles is artificially increased. The decrease **in** the total number of particles as a function of time is described by the following equation:

$$-\frac{dn}{dt} = \frac{4}{3} \cdot n_1 \cdot n_2 \cdot R^3 \cdot G_v$$

in which:
$G_v$ = velocity gradient for floc formation [$s^{-1}$]
R = collision radius [m]
$n_1$ = number of particles with diameter $d_1$ [-]
$n_2$ = number of particles with diameter $d_2$ [-]

The collision radius is defined by $0.5 \cdot (d_1 + d_2)$. Assuming that all particles have the same diameter, the equation can be rewritten as:

$$-\frac{dn}{dt} = \frac{4 \cdot \alpha \cdot n^2 \cdot d^3 \cdot G_v}{3}$$

For spherical particles the volumetric concentration is described as:

$$c_v = \frac{1}{6} \cdot \pi \cdot n \cdot d^3$$

Deriving $n \cdot d^3$ and substituting it:

$$-\frac{dn}{dt} = \frac{8 \cdot \alpha \cdot n \cdot c_v \cdot G_v}{\pi}$$

resulting in a solution for plug flow:

$$\frac{n}{n_o} = e^{-k_a \cdot c_v \cdot G_v \cdot t}$$

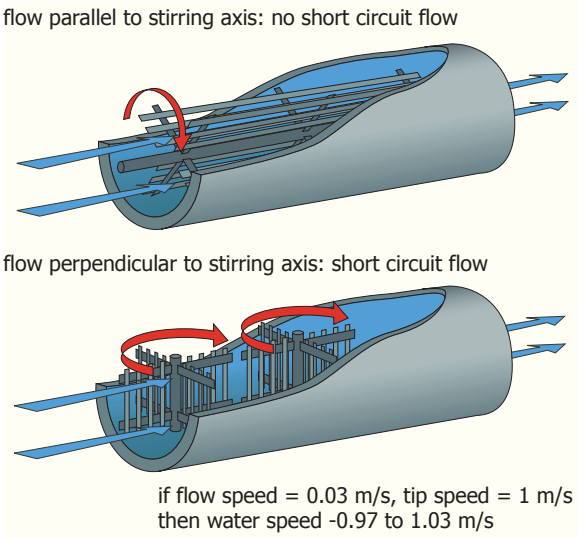



*Figure 18 - Mixers in line and perpendicular to the flow direction of the water*

and for completely mixed systems:

$$\frac{n}{n_o} = \frac{1}{1 + k_a \cdot c_v \cdot G_v \cdot t}$$

With these equations it can be calculated that orthokinetic floc formation of particles with a diameter of 1 µm only takes place when velocity gradients higher than 10 $s^{-1}$ are applied. Otherwise, perikinetic floc formation is predominant.

## *3.2 Floc formation in practice*

Parameters that are important to the design of a floc formation installation are the following:
- residence time T
- residence time distribution
- velocity gradient for floc formation $G_v$
- floc volume concentration $c_v$.

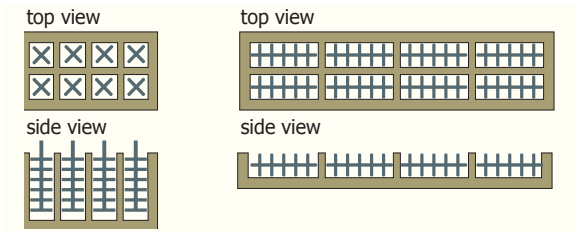



*Figure 19 - Plug flow mixing systems for floc formation, mixer in line with water flow*

**Residence time**

Time is needed for the formation of removable flocs. The applied residence time varies between 500 and 3600 sec. On average the residence time for floc formation is about 30 minutes.
To determine the required residence time, jar-test experiments are carried out.

**Residence time distribution**

When water flows through a tank, the residence time of every droplet is different. For some droplets the residence time is longer and for others shorter than the average. The consequence is that, in practice, the floc formation process performs worse than can be expected, based on theory.
In order to approach the perfect plug flow where every droplet has the same residence time, criteria are developed for the design of a floc formation chamber. A plug flow can be approached when the ratio between the length and width of a tank is at least equal to 3.

Mixers in floc formation chambers take care of the dispersion of energy and collision of the particles. It is, however, important that that the mixers be in line with the flow direction (Figure 18).
If the mixers are placed perpendicular to the flow direction, some water droplets are accelerated and other are slowed down, resulting in a larger residence time distribution.
When the axis of the mixer is in line with the flow, the influence is limited.

In two different floc formation systems the length/width ratio of 3 and the direction of the mixers are

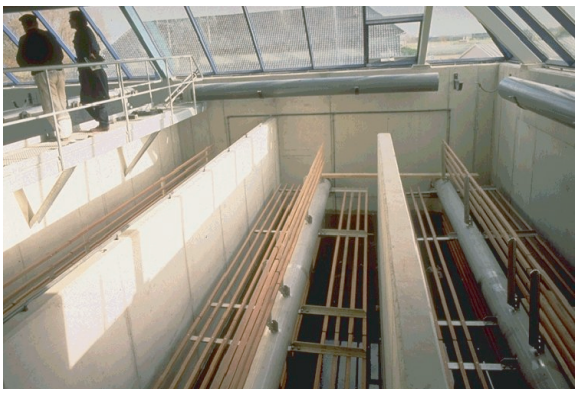


*Figure 20 - Mixing device*

considered:
- horizontal, long and narrow (Figure 19 right).
- vertical, deep and narrow chambers (Figure 19 left).

**Velocity gradient**

After coagulation the colloids and humic acid are destabilized and many small particles are present in the water.

Mixers that are placed in the floc formation chambers dissipate energy in the water (Figure 20), resulting in the collision of neutral particles and the formation of flocs.

The degree of energy dissipation is expressed, like for coagulation, in the velocity gradient. The velocity gradient is mainly created by mixers.

Alternatively, hydraulic floc formation can be applied where the head loss between two chambers delivers the energy for the formation of flocs. The drawback of hydraulic flocculation is the uneven energy input.

The velocity gradient for floc formation is expressed in the parameter $G_v$ and is defined by:

$$G_v = \sqrt{\frac{P}{\mu \cdot V}}$$

The energy dissipation from the mixers can be calculated with the following equation:

$$P = \rho_w \cdot \pi^3 \cdot (1 - k_2)^3 \cdot N^3 \cdot \sum (c_d \cdot L_{blade} \cdot (r_u^4 - r_i^4))$$

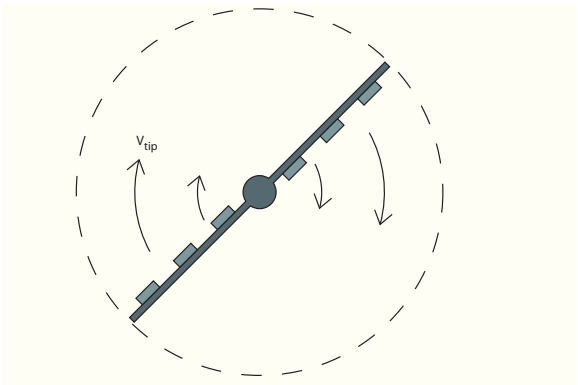



*Figure 21 - Tip velocity*

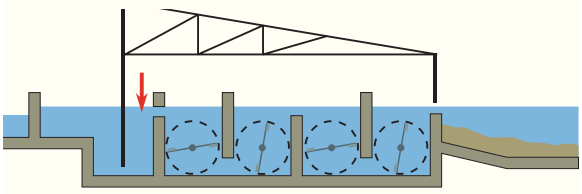


*Figure 22 - Floc formation installation WRK I/II*

in which:

| | | | |
|---|---|---|---|
| $k_2$ | = | constant≈ 0.25 | [-] |
| N | = | rotation speed | [rotations/min] |
| $C_d$ | = | constant ≈ 1.50 | [-] |
| $L_{blade}$ | = | length of mixer blade | [m] |
| $r_u$ | = | distance from exterior of mixing blade to axis | [m] |
| $r_i$ | = | distance from interior of mixing blade to axis | [m] |

According to the formula for the dissipation energy from the mixer, the rotation speed is the only operation parameter. The other parameters are already determined during the design process.

The velocity gradient in operation can thus be calculated by:

$$G_v = \text{const.} \cdot \sqrt{N^3}$$

The calculated velocity gradient is the average in the floc formation chamber.

The velocity of the mixing blade in the floc formation chamber depends on the radius and the rotation speed. The velocity is greatest at the tip of the mixing blade and is called the tip velocity

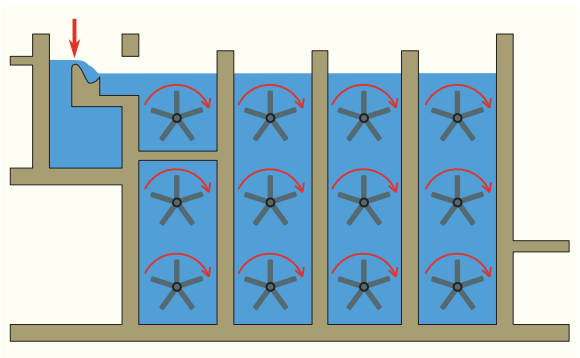


*Figure 23 - Floc formation installation WRK III, division in different compartments*

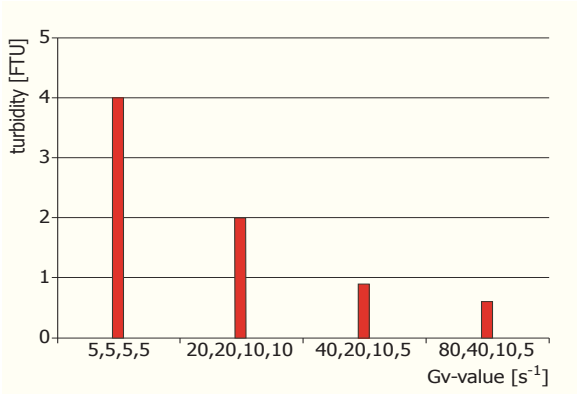



*Figure 24 - Influence of G-value on floc formation in different compartments*

(Figure 21) and can be calculated by:

$$v_{tip} = 2 \cdot \pi \cdot r \cdot N$$

The higher the rotation speed, the higher the tip velocity. When the tip velocity is higher than 1 m/s, formed flocs are broken up.
When the rotation speed of a mixer is known, the maximum radius of a mixing blade can be determined. A rotation speed of 4 rotations per minute allows a maximum radius of 2.4 meters.

During coagulation small neutral particles are formed and grow, after collision, into removable flocs.
To increase the collision frequency of the particles, a high mixing intensity must be applied. The small particles collide and larger particles are formed, but in the mean time, the risk of floc break-up increases as a result of fluid shear.
Therefore, the floc formation chamber is divided

*Figure 25 - Hydraulic floc formation*

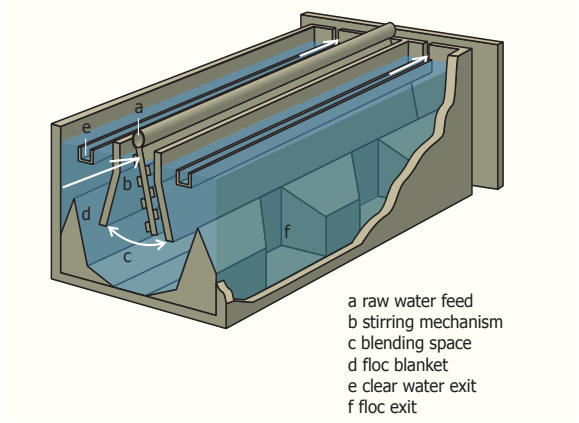



*Figure 26 - Floc blanket installation at Berenplaat*

into several compartments (Figure 22 and 23) with decreasing velocity gradients (Figure 24).
In the first compartment the velocity gradient will be high (about 100 $s^{-1}$) and in the last compartment the velocity gradient will be low (about 5 $s^{-1}$). The optimal operation of the mixers must be determined empirically.
The flow opening between the compartments must be large enough to avoid local energy dissipation, as is the case in hydraulic flocculation (Figure 25).

**Floc volume concentration**

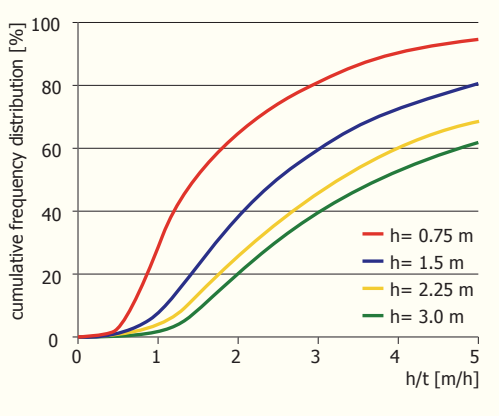
The floc volume concentration is of importance when the raw water contains low concentrations of suspended solids and a small amount of flocs is formed.
To stimulate the formation of new flocs, part of the formed flocs are recirculated, resulting in shorter residence times and lower velocity gradients.

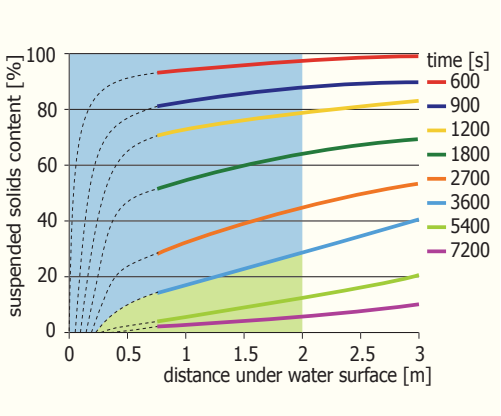
*Figure 27 - Mixing device floc blanket installation at Berenplaat*

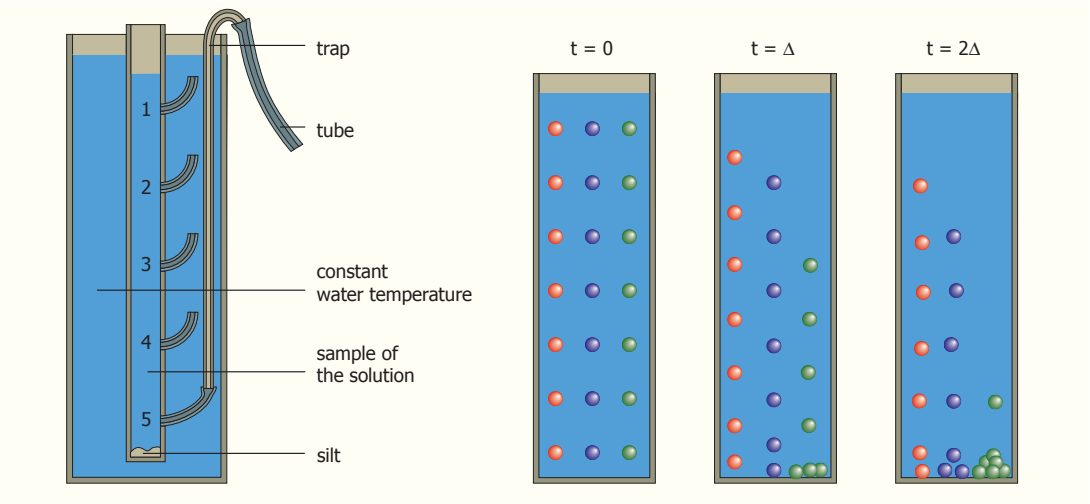
*Figure 28 - Mixing device flow blanket installation at Berenplaat*

WATER TREATMENT

# Sedimentation



**Framework**

This module represents sedimentation.

**Contents**

This module has the following contents:

1. Introduction
2. Theory
   - 2.1 Sedimentation of discrete particles
   - 2.2 Horizontal flow settling tanks in practice
   - 2.3 Settling efficiency of a suspension
3. Influences on settling in a horizontal flow tank
   - 3.1 Influence of turbulence
   - 3.2 Influence of stability
   - 3.3 Influence of bottom scour
   - 3.3 Influence of flocculant settling
4. Practice
   - 4.1 Determination of the dimensions of an ideal settling tank
   - 4.2 Inlet constructions
   - 4.3 Outlet constructions
5. Settling tank alternatives
   - 5.1 Vertical flow settling tank
   - 5.2 Floc blanket clarifier
   - 5.3 Tray settling tanks
   - 5.4 Tilted plate settling

# 1 Introduction

Sedimentation is a treatment process in which suspended particles, like flocs, sand and clay are re-moved from the water.
Sedimentation can take place naturally in reservoirs or in compact settling installations.
Examples of settling installations are the horizontal flow settling tanks, the tilted plate settlers and the floc blanket installations.

Sedimentation is frequently used in surface water treatment to avoid rapid clogging of sand filters after coagulation and floc formation (Figure 1).
Sedimentation is applied in groundwater treatment installations for backwash water treatment.

In horizontal flow settling tanks (Figure 2) water is uniformly distributed over the cross-sectional area of the tank in the inlet zone.
A stable, non-turbulent, flow in the settling zone takes care of the settling of suspended matter in the settling zone.
The sludge accumulates on the bottom or is continuously removed.
In the outlet zone the settled sludge must be prevented from being re-suspended and washed out with the effluent.

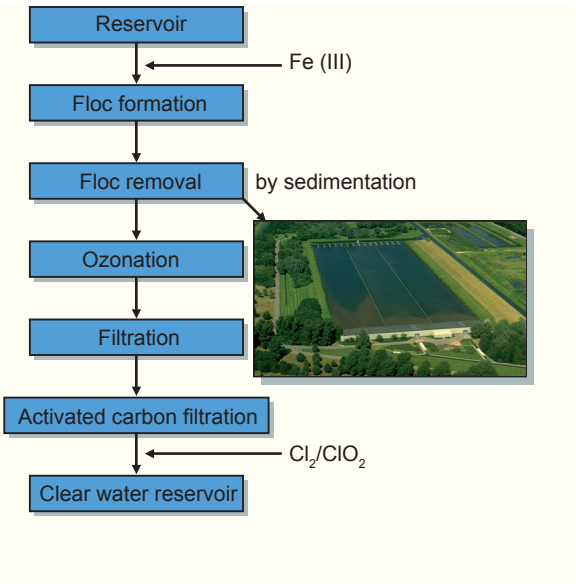



*Figure 1 - Process scheme of a surface water treatment plant*

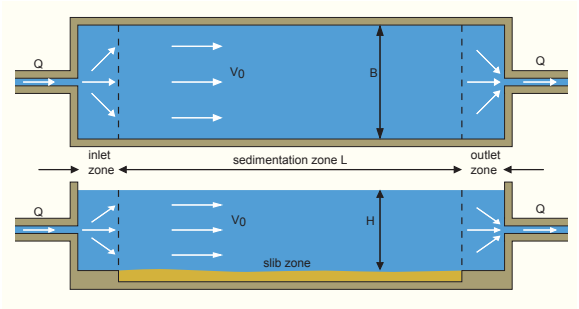



*Figure 2 - Horizontal flow settling tank*

Sedimentation occurs because of the difference in density between suspended particles and water.
The following factors influence the sedimentation process: density and size of suspended particles, water temperature, turbulence, stability of flow, bottom scour and flocculation:

- density — the greater the density of the particles, the faster the particles settle
- size — the larger the particles are, the faster they settle
- temperature — the lower the temperature of the water is, the higher the viscosity, so the slower the particles settle
- turbulence — the more turbulent the flow is, the slower the particles settle
- stability — instability can result in a short-circuit flow, influencing the settling of particles
- bottom scour — during bottom scour, settled particles are re-suspended and washed out with the effluent
- flocculation — flocculation results in larger particles, increasing the settling velocity.

# 2 Theory

## *2.1 Sedimentation of discrete particles*

Discrete particles do not change their size, shape or weight during the settling process (and thus do not form aggregates).
A discrete particle in a fluid will settle under the influence of gravity. The particle will accelerate

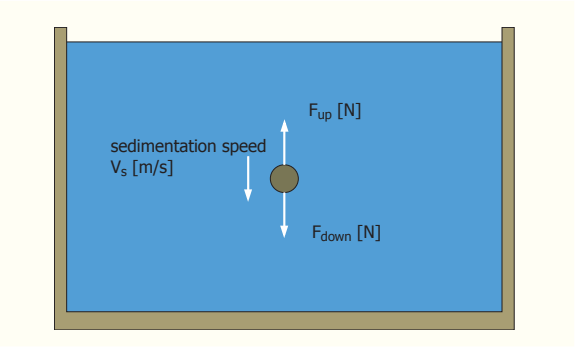



*Figure 3 - Forces on a settling particle*

until the frictional drag force of the fluid equals the value of the gravitational force, after which the vertical (settling) velocity of the particle will be constant (Figure 3).

The upward directed force on the particle, caused by the frictional drag of the fluid, can be calculated by:

$$F_{up} = c_D \cdot \frac{\rho_w}{2} \cdot v_s^2 \cdot A$$

in which:

| | | | |
|---|---|---|---|
| $F_{up}$ | = | upward directed force by friction | [N] |
| $c_D$ | = | drag coefficient | [-] |
| $\rho_w$ | = | density of water | [kg/m$^3$] |
| $v_s$ | = | settling velocity | [m/s] |
| A | = | projected area of the particle | [m$^2$] |

The downward directed force, caused by the difference in density between the particle and the water, can be calculated by:

$$F_{down} = (\rho_s - \rho_w) \cdot g \cdot V$$

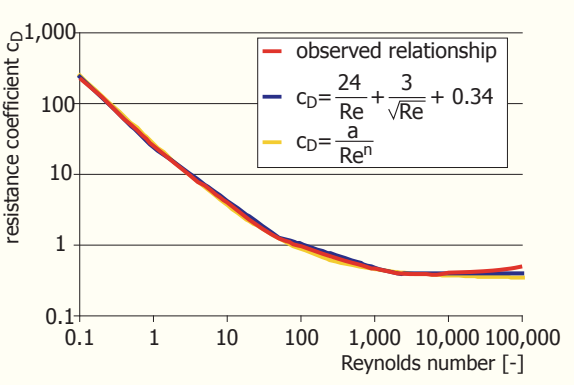



*Figure 4 - Relationship between Reynolds number and drag coefficient*

in which:

| | | | |
|---|---|---|---|
| $F_{down}$ | = | downward directed flow by gravity | [N] |
| $\rho_s$ | = | specific density of particle | [kg/m$^3$] |
| g | = | gravity constant | [m/s$^2$] |
| V | = | volume of particle | [m$^3$] |

Equality of both forces, assuming a spherical particle, gives as the settling velocity:

$$v_s = \sqrt{\frac{4}{3 \cdot c_D} \cdot \frac{\rho_s - \rho_w}{\rho_w} \cdot g \cdot d}$$

in which:

| | | | |
|---|---|---|---|
| d | = | diameter of spherical particle | [m] |

The settling velocity is thus dependent on:
- density of particle and fluid
- diameter (size) of particle
- flow pattern around particle.

The flow pattern around the particle is incorporated in the drag coefficient. The value of the drag coefficient is not constant, but depends on the magnitude of the Reynolds number for settling. For spherical particles the Reynolds number is given by:

$$Re = \frac{v_s \cdot d}{\nu}$$

in which:

| | | | |
|---|---|---|---|
| $\nu$ | = | kinematic viscosity | [m$^2$/s] |

In drinking water treatment practice, laminar settling normally occurs. The Reynolds number for laminar settling of spheres is Re<1, resulting in the following relationship between the Reynolds number and the drag coefficient:

Substitution of this relationship in the equation for the settling velocity gives the Stokes' equation:

$$v_s = \frac{1}{18} \cdot \frac{g}{\nu} \cdot \frac{\rho_s - \rho_w}{\rho_w} \cdot d^2$$

The settling velocity is thus dependent on the viscosity of the fluid and also the temperature.

The relationship between kinematic viscosity and temperature is:

$$v = \frac{497 \cdot 10^{-6}}{(T + 42.5)^{1.5}}$$

in which:

T = temperature [$^oC$]

When the Reynolds number Re > 1600, settling is turbulent and when 1<Re<1600, settling is in transition between laminar and turbulent.
In Figure 4 the relationship between the drag coefficient and the Reynolds number is represented.

In Figure 5 the settling velocity as a function of particle size and density is shown.

## 2.2 *Horizontal flow settling tanks in practice*

In practice, settling occurs in flowing water. An ideal horizontal flow settling tank has the following characteristics:

- at the inlet the suspension has a uniform composition over the cross-section of the tank
- the horizontal velocity $v_o$ is the same in all parts of the tank
- a particle that reaches the bottom is definitively removed from the process.

The flow velocity in a horizontal settling tank is:

$$v_o = \frac{Q}{B \cdot H}$$

in which:

$v_o$ = horizontal flow velocity [m/h]
Q = flow [$m^3$/h]
B = width of the tank [m]
H = height of the tank [m]

The surface loading of a settling tank is determined by:

$$q = \frac{Q}{B \cdot L}$$

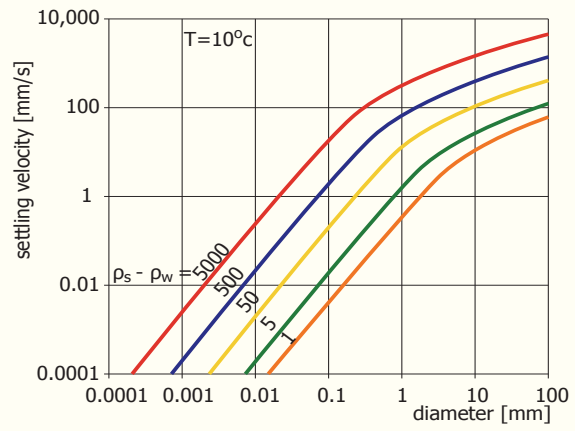



*Figure 5 - Settling velocity of discrete spherical particles*

in which:

q = surface loading [$m^3/(m^2 \cdot h)$]
L = length of the tank [m]

In Figure 6 the trajectory of a particle is represented. After $t_1$ the water leaves the tank and after $t_2$ the particle is settled. The particles will settle, therefore, when $t_2 < t_1$.
The velocity of the particle is divided into horizontal and vertical components and the settling times can be written as:

$$t_2 \le t_1 \Rightarrow \frac{H}{v_s} \le \frac{L}{v_0} \Rightarrow \frac{H}{v_s} \le \frac{B \cdot H \cdot L}{Q}$$

$$\Rightarrow \frac{1}{v_s} \le \frac{1}{q} \Rightarrow v_s \ge q$$

In special cases, when the settling velocity equals the surface loading, the particle reaches the end of the tank. This settling velocity is called the critical velocity $v_{so}$.
It can be concluded that a particle will only be removed if the settling velocity is greater than or equal to the critical settling velocity (Figure 7).

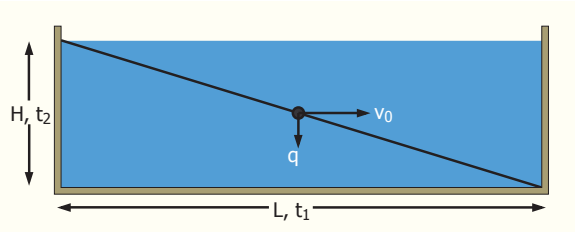



*Figure 6 - Settling in a horizontal flow settling tank*

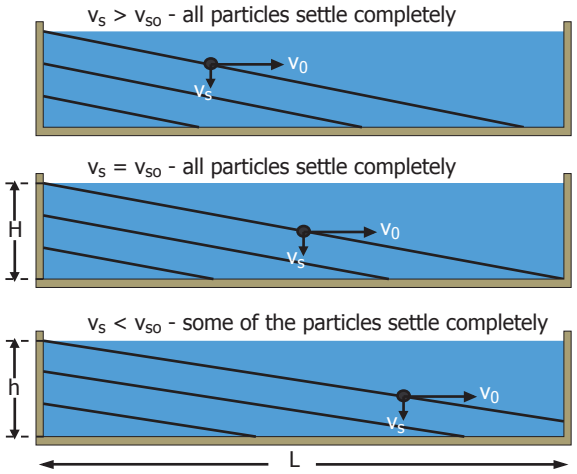



*Figure 7 - Settling of a suspension in a horizontal flow*

After determining the settling velocity of a particle during a settling test, the surface loading and thus the dimensions of the tank can be determined.
It is remarkable that, in theory, settling in a horizontal flow settling tank is only determined by the flow and the surface area of the tank and is independent on the height of the tank.

The fraction of the particles that settle in case $v_s < v_{so}$ is (Figure 7):

$$\frac{h}{H} = \frac{v_s \cdot T}{v_{so} \cdot T} = \frac{v_s}{v_{so}}$$

in which:
T = residence time of water in the settling tank[s]

The residence time of water in the settling tank is expressed as T and equals $t_1$ from Figure 6.

### 2.3 Settling efficiency of a suspension

In a suspension the fraction of particles with a settling velocity higher than the surface loading settle completely. The fraction with a lower settling velocity settles partly. The efficiency is determined from the cumulative frequency distribution of settling velocities obtained from a settling test.

The settling test is executed in a cylindrical container (column) filled with a homogeneous sample of the suspension to be tested (Figure 8). At different time intervals samples are taken at different depths and analyzed for suspended solids, turbidity or any other index that can be reduced by settling. The depth is measured with the water surface as reference. In Table 1 the analyses of a settling column test at depth h=1.0 m are represented (Figure 8).

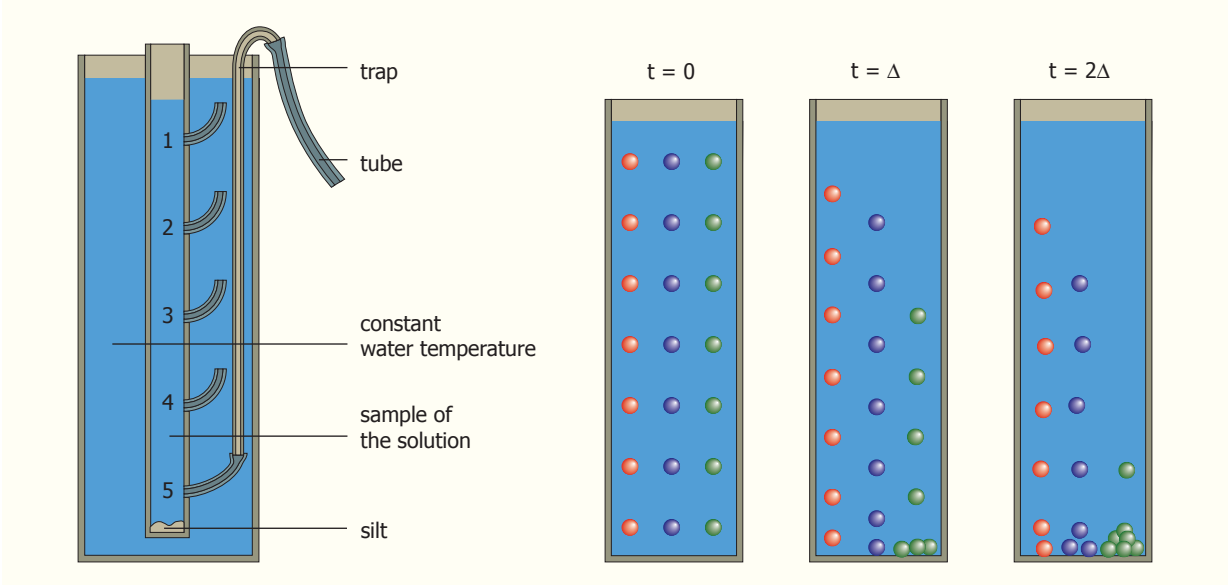



*Figure 8 - Settling column and representation of different settling velocities*

*Table 1 - Particle concentration and relative particle concentration from a settling test at a depth of h = 1.0 m*

| t (s) | 0 | 666 | 900 | 1800 | 2700 | 3600 | 5400 | 7200 |
|---|---|---|---|---|---|---|---|---|
| c (ppm) | 86 | 84 | 79 | 57 | 41 | 29 | 7 | 3 |
| p=c/co (%) | 100 | 98 | 92 | 66 | 48 | 34 | 8 | 4 |

In Figure 9 the cumulative frequency distribution of the settling velocities is represented. The ratio of sampling depth and time is given as a function of the relative solids concentration. The solids with the lowest settling velocity determine the residence time of a settling system.

The particles with a settling velocity higher than the critical settling velocity $v_{so}$ are removed completely. This is represented in Figure 9 by the red arrow. Expressing the relative solids concentration for a settling velocity of $v_{so}$ as $p_o$, the first part of the settling efficiency is:

$$r_1 = 1 - p_o$$

in which:

$r_1$ = part of the efficiency caused by complete settling [-]

$p_o$ = relative solids concentration at surface loading $s_o$ [%]

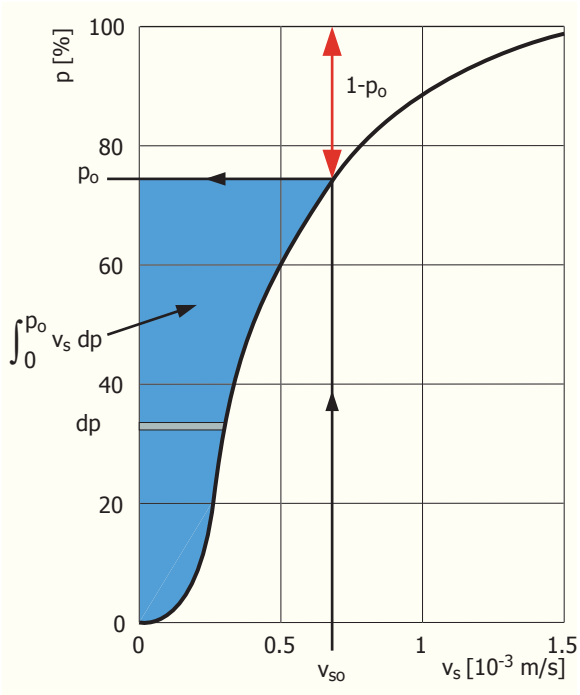



*Figure 9 - Cumulative frequency distribution of settling velocities*

From the particles with a lower settling velocity than $v_{so}$, only the particles that enter the tank at a reduced height will be removed.

From the fraction of particles dp with settling velocity $v_s$, only the fraction h/H or $v_s/vs_o$ will be removed. This part of the efficiency (partial removal) can be described by:

$$r_2 = \int_0^{p_o} \frac{v_s}{v_{so}}\,dp = \frac{1}{v_{so}} \int_0^{p_o} v_s\,dp$$

in which:

$r_2$ = part of the efficiency caused by partial settling [-]

The efficiency caused by partial settling is represented by the blue surface in Figure 9 divided by the critical settling velocity. Graphically, this part of the total efficiency can be determined as shown in Figure 10.

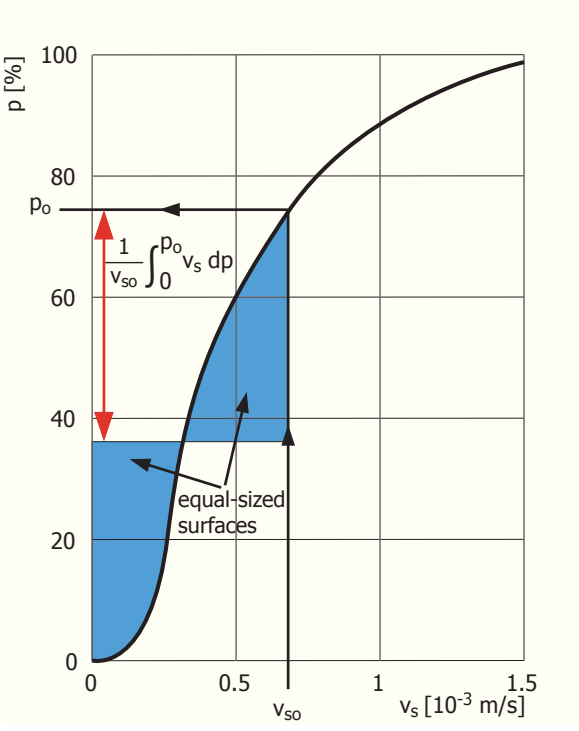



*Figure 10 - Efficiency of partial settling*

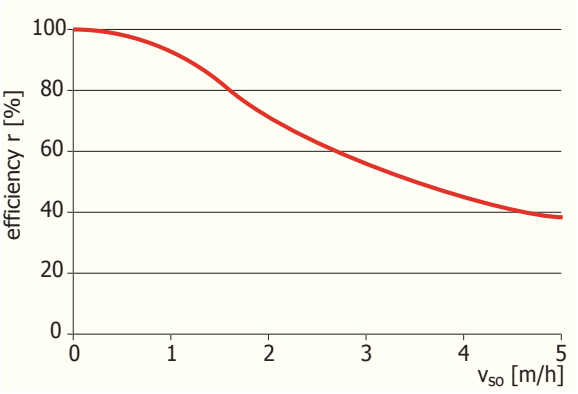



*Figure 11 - Removal efficiency in a horizontal flow settling tank*

The equation of the total settling efficiency becomes:

$$r = (1 - p_o) + \frac{1}{v_{so}} \int_0^{p_o} v_s \, dp$$

For different values of $v_{so}$ the efficiency is calculated and the results are represented in Figure 11.
It can be concluded that with increasing surface loading of the settling tank (by increasing flow), the settling efficiency decreases.

## 3 Influences on settling in a horizontal flow tank

In the preceding paragraphs an ideal flow and discrete settling were assumed.
In practice, however, the ideal situation does not exist and the efficiency is influenced by:
- turbulence of flow
- instability of flow
- bottom scour
- flocculation.

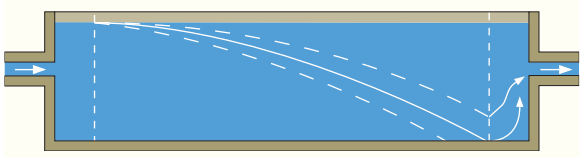


*Figure 12 - Influence of turbulence on settling in a horizontal flow settling tank*

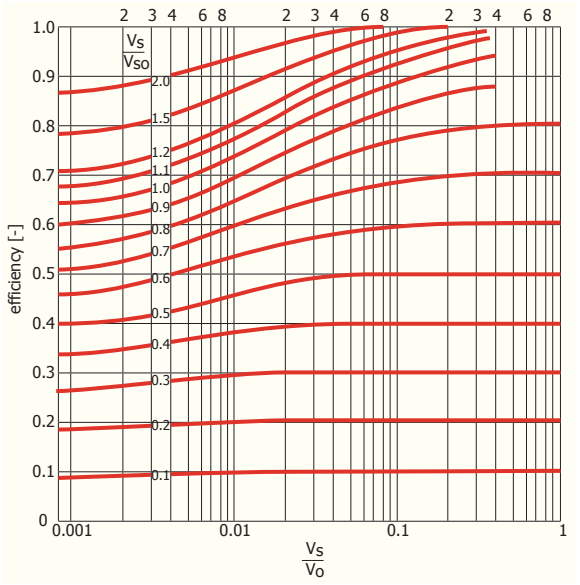



*Figure 13 - Influence of turbulence on the efficiency of settling*

### 3.1 Influence of turbulence

In laminar flow in a horizontal flow tank, a particle follows a straight line.
In turbulent flow, eddies will transport particles in a random direction, influencing the settling of the particles (some settle faster and others slower) (Figure 12).

With the Reynolds number the flow characteristics can be determined:
- laminar flow: Re < 2000
- turbulent flow: Re > 2000.

The Reynolds number for flow in a tank can be calculated with:

$$Re = \frac{v_o \cdot R}{\nu}$$

in which:
R = hydraulic radius of a settling tank [m]

The hydraulic radius of a rectangular tank can be calculated with:

$$R = \frac{B \cdot H}{B + 2 \cdot H}$$

With the expression $v_o$=Q/(B•H) the Reynolds

number can be rewritten as:

$$Re = \frac{Q}{\nu} \cdot \frac{1}{B + 2 \cdot H}$$

In Figure 13 the settling efficiency for turbulent flow is represented as a function of $v_s/v_{so}$ and $v_s/v_o$.

In practice turbulence is not always a disadvantage because, in general, flocculant settling occurs (section 3.4). Turbulence increases the collision frequency of particles, thus increasing the efficiency of the flocculant settling.

### 3.2 Influence of stability

Flow is called stable when short circuiting does not occur.
In Figure 14 an example of a short-circuit flow caused by wind effects is illustrated. The wind creates a dead zone (or eddy) in the corner of the settling tank. The water flow can then flow, locally, in the opposite direction from the general flow through the tank.

Stability of flow is characterized by the Camp number $c_p$:

$$c_p = \frac{v_o^2}{g \cdot R}$$

Substituting the equations for vo and R for a rectangular tank, the Camp number becomes:

$$c_p = \frac{Q^2}{g} \cdot \frac{B + 2 \cdot H}{B^3 \cdot H^3}$$

$c_p > 1 \cdot 10^{-5}$ stable flow
$c_p < 1 \cdot 10^{-5}$ unstable flow

In Figure 15 the minimal residence time ($T_i$) and the average residence time ($T_a$) of water droplets are represented in comparison with the theoretical residence time ($T_o$) for different values of the Camp number.

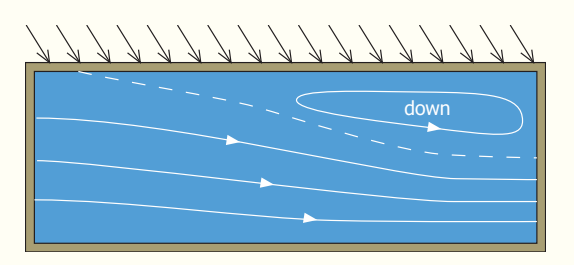



*Figure 14 - Short-circuit flow caused by wind*

From Figure 15 it can be concluded that the lower the Camp number is (and thus more short-circuit flow occurs), the shorter the minimal and average residence times become. This is due to the decrease in the effective cross-section of the settling tank and, therefore, to an increase in flow velocity.
The efficiency of a settling tank, therefore, will be lower than is the case in a stable flow condition.

### 3.3 Influence of bottom scour

In theory, a particle is removed from the water when it reaches the bottom of the settling tank.
In practice, however, it is possible that resuspension of already settled particles occurs.

In Figure 16 the forces on particles at the bottom of the tank are shown.
The shear force of water on a spherical particle is:

$$\tau = \frac{\lambda}{8} \cdot \rho_w \cdot v_{sc}^2$$

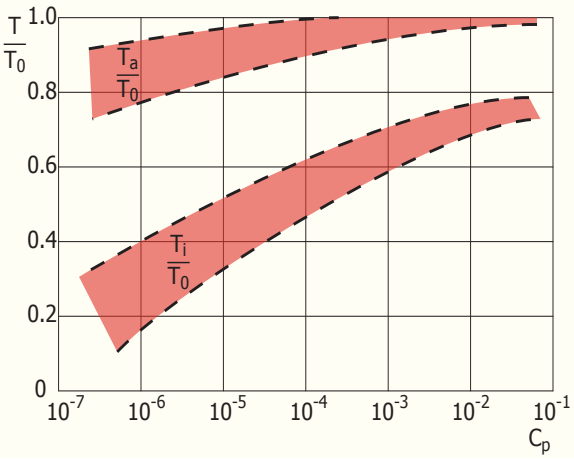



*Figure 15 - Short-circuit flow*

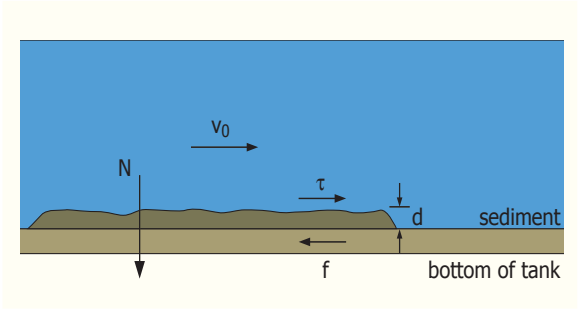



*Figure 16 - Bottom scour*

in which:

| | | | |
|---|---|---|---|
| $\tau$ | = | hydraulic shear | [N/m$^2$] |
| $\lambda$ | = | hydraulic friction factor ($\lambda$ = 0.03) | [-] |
| $v_{sc}$ | = | critical scour velocity | [m/s] |

The shear force of particles at the bottom (mechanical friction) is proportional to the submerged weight of the sludge layer:

$$f = \beta . (\rho_s - \rho_w) . g . d$$

in which:

| | | | |
|---|---|---|---|
| f | = | mechanical shear | [N/m$^2$] |
| $\beta$ | = | mechanical shear factor (b = 0.05) | [-] |

In equilibrium the hydraulic shear equals the mechanical shear and the critical scour velocity can be calculated:

$$v_{sc} = \sqrt{\frac{40}{3} \cdot \frac{\rho_s - \rho_w}{\rho_w} \cdot g \cdot d}$$

When the flow velocity in a settling tank is lower than the scour velocity, bottom scour will not occur:

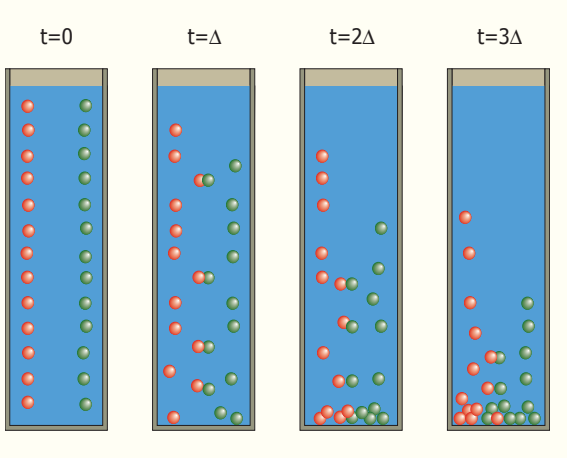



*Figure 17 - Flocculant settling*

$v_0 <= v_{sc}$ no bottom scour

Given the surface loading, the width and depth of a settling tank can be determined based on this criterion.

### 3.4 Influence of flocculant settling

During settling, aggregates are formed as a result of collisions between particles, and settling velocities will increase. This phenomenon is called flocculant settling (Figure 17).

In Table 2 the results of a settling test of a flocculant suspension are shown..

In Figure18 the cumulative frequency distribution of settling velocities is given at different tank depths. From the fact that the distributions differ

*Table 2 - Relative particle concentration from a settling test*

| | h = 0.075 m | h = 1.5 m | h = 2.25m | h = 3.0 m |
|---|---|---|---|---|
| t = 0 s | 100 | 100 | 100 | 100 |
| t = 600 s | 93 | 96 | 98 | 99 |
| t = 1200 s | 81 | 86 | 88.5 | 89.5 |
| t = 1800 s | 70.5 | 77.5 | 81 | 83 |
| t = 2700 s | 28 | 38 | 46.5 | 53 |
| t = 3600 s | 13.5 | 22 | 31 | 40 |
| t = 5400 s | 3 | 8 | 13.5 | 20 |
| t = 7200 s | 1.5 | 3 | 6 | 9.5 |

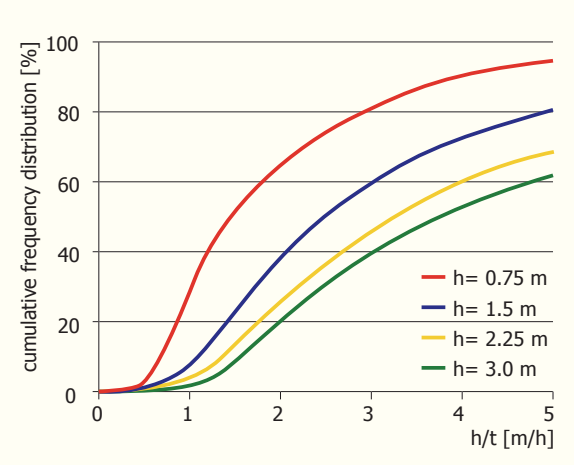



*Figure 18 - Cumulative frequency distribution of settling velocities at different tank depths*

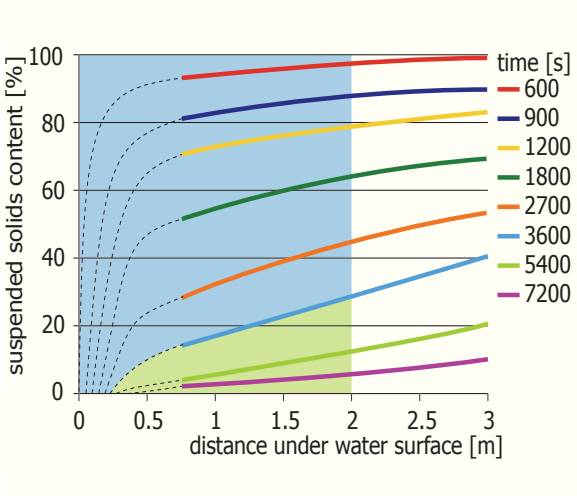



*Figure 19 - The relative particle concentration*

**Example**

A horizontal flow settling tank has a height of 2 m, a width of 20 m and a length of 45 m. The flow through the tank is 0.5 $m^3/s$ and the water temperature is 10 $^0C$

Check if the tank meets the hydraulic requirements.

The horizontal flow velocity and the critical settling velocity are:

$$v_o = \frac{Q}{B \cdot H} = \frac{0.5}{20 \cdot 2} = 12.5 \cdot 10^{-3} m/s$$

$$v_{SO} = \frac{Q}{B \cdot L} = \frac{0.5}{20 \cdot 45} = 0.56 \cdot 10^{-3} m/s$$

The hydraulic radius of the tank can be calculated with:

$$R = \frac{B \cdot H}{B + 2 \cdot H} = \frac{20 \cdot 2}{20 + 2 \cdot 2} = 1.67$$

The Reynolds number and the Camp number are:

$$Re = \frac{v_o \cdot R}{\nu} = \frac{12.5 \cdot 10^{-3} \cdot 1.67}{1.31 \cdot 10^{-6}} = 15935$$

$$c_p = \frac{v_o^2}{g \cdot R} = \frac{\left(12.5 \cdot 10^{-3}\right)^2}{9.81 \cdot 1.67} = 0.954 \cdot 10^{-5}$$

The Reynolds number is higher than 2000 and the flow will be turbulent. The Camp number is about $1 \cdot 10^{-5}$ and no short-circuit flow will occur.

Determine the efficiency of the settling tank considering the suspension from the settling test in Table 2.

In Figure 19 the suspended solids content is represented as a function of water depth at the different sampling times from Table 2. Between 0 and 0.75 meters the progress of the graph is estimated (dotted line). The residence time is:

$$\tau = \frac{B \cdot H \cdot L}{Q} = \frac{20 \cdot 2 \cdot 45}{0.5} = 3600s$$

From Figure 19 the efficiency can be determined, assuming a residence time of 3600 seconds and a height of 2 meters. The blue surface above the line indicates the amount of solids that are settled. The green surface below the line indicates the amount of solids that are still in suspension after 3600 seconds. It can be concluded after measuring the surfaces that the settling efficiency is 80%.

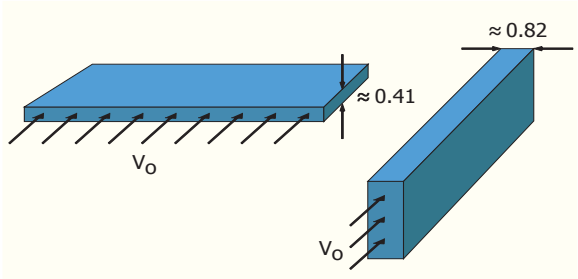



*Figure 20 - Settling tanks with laminar and stable flows*

over the height of the tank, it can be concluded that flocculant settling occurs.

From Figure 19 it can also be concluded that the efficiency increases with the increasing depth.
For flocculant settling, in contrast to discrete settling, the height of the tank is of importance to the settling efficiency.

# 4 Practice

## 4.1 Determination of the dimensions of an ideal settling tank

In ideal settling tanks the flow is stable ($c_p > 10^{-5}$) without turbulence (Re < 2000).
At a temperature of 10°C these conditions are met with a horizontal flow velocity and a hydraulic radius of:
$v_o = 6.4 \cdot 10^{-3}$ m/s
R < 0.41 m

Tanks that meet these conditions are short, wide and shallow or long, narrow and deep (Figure 20).
These constructions, however, are expensive due to the amount of space they occupy.
In practice, a tank will be a compromise between the Reynolds and Camp numbers, on the one hand, and the construction costs, on the other, limiting the length/width/depth ratios.

## 4.2 Inlet constructions

In the preceding paragraphs it was assumed that the water is uniformly distributed over the cross-section of the tank, but in practice this assumption is not totally accurate.
For an even distribution of the water over the width (and depth) of the tank, inlet constructions are introduced.
In Figure 21 an example of an inlet construction is represented. The inlet velocity is reduced by introducing several inlet channels, followed by a diffuser wall that distributes the water over the entire cross-section of the tank.

A diffuser wall (Figure 22) has openings to distribute the water over the width (and depth) of the tank. At the end of the wall, the flow velocity in the inlet channel is zero and so is the velocity head. The head loss caused by friction, however, is lower than the decrease in velocity head, resulting in an increase in the piezometric level.
The water level at the end of the inlet channel is, thus, higher than the level at the beginning. The result is that at the end of the inlet channel more water enters the tank than at the beginning of the inlet channel. To avoid this uneven distribution, the head loss over the openings in the diffuser wall must be larger than the difference in piezometric level induced by the decrease in flow

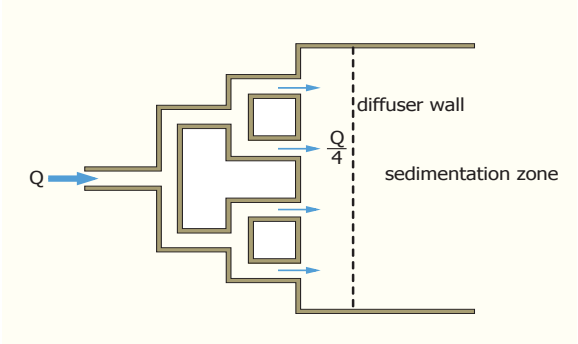



*Figure 21 - Inlet construction*

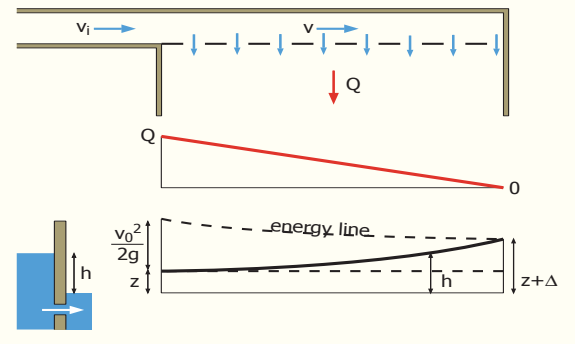



*Figure 22 - Diffuser wall*

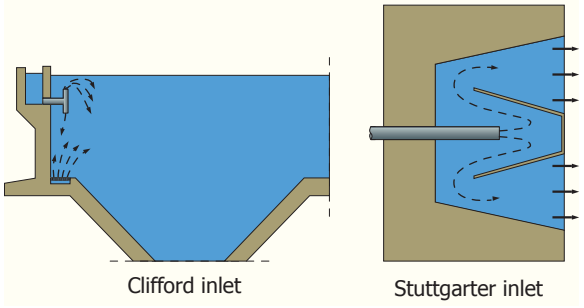



*Figure 23 - Clifford and Stuttgarter inlet*

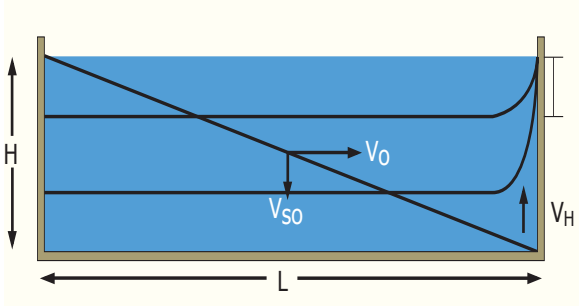



*Figure 24 - Upward velocity to overflow weir*

velocity.

In practices, more as well as alternative inlet constructions exist, like the Clifford and the Stuttgarter inlet (Figure 23).

### 4.3 Outlet constructions

The outlet construction is situated at the end of the settling tank and generally consists of an overflow weir.
At the outlet construction, re-suspension of settled solids must be prevented and the flow velocity in an upward direction will thus be limited (Figure 24).

The flow velocity in an upward direction is:

$$v_H = \frac{1}{5} \cdot \frac{Q}{B \cdot H} < v_{so}$$

in which:
$v_H$ =outflow velocity in an upward direction [m/s]

Resulting in:

$$\frac{L}{H} < 5$$

Most horizontal flow settling tank have an L/H>5 and thus:

$$\frac{Q}{n \cdot B} < 5 \cdot H \cdot v_{so}$$

Therefore, the length of the overflow weir must be several times the width of the tank.
To create sufficient length for the overflow weir, several troughs are placed parallel to each other(Figure 25).

### 4.4 Sludge zone and removal

In the sludge zone the solids are accumulated.
The removal of the sludge can be done hydraulically and mechanically.

Hydraulic sludge removal is done at regular intervals by dewatering the tank and flushing the sludge with pressured water (from hydrants) to a hopper at the bottom of the tank from where it is removed by gravity or by pumping.

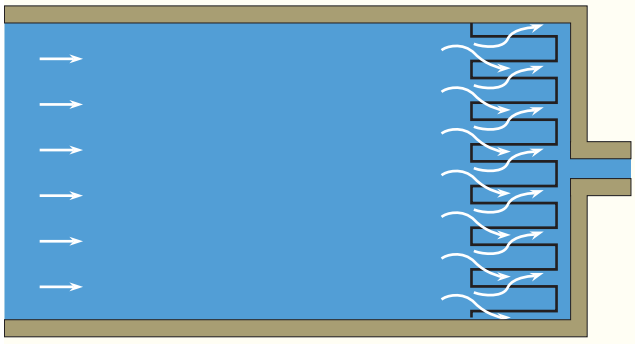


*Figure 25 - Overflow weir for effluent discharge*

Mechanical sludge removal is frequently applied when sludge volumes are large or the sludge is unstable, resulting in anaerobic decomposition during storage in the sludge zone.
Mechanical sludge removal consists of scrapers that transport the sludge to a hopper in the middle of a round settling tank or near the inlet of a rectangular tank. From the hopper, the sludge is removed.

## 5 Settling tank alternatives

### *5.1 Vertical flow settling tank*

In vertical flow settling tanks the inlet of the water to be treated is situated at the bottom of the tank and the water flows in an upward direction (Figure 26).
The flow velocity equals, in this case, the surface loading:

$$v_o = \frac{Q}{B \cdot L} = s_o$$

The result is that only particles with a settling velocity higher than the upflow velocity will settle and others will be washed out:

$s >= s_0$ settles completely

$s < s_0$ does not settle

The settling efficiency is entirely determined by the particles that settle completely (see Figure 9):

$$r = 1 - p_o$$

The settling efficiency of discrete particles in vertical flow settling tanks is lower than in horizontal flow tanks, and vertical flow tanks are therefore not used for discrete, totally flocculated, suspensions.
In the case of flocculant settling, vertical flow tanks are used (e.g., in the form of floc blanket clarifiers).

### *5.2 Floc blanket clarifier*

The floc blanket clarifier consists of a (conical) vertical flow tank (Figure 27).
Coagulant is dosed at the inlet of the clarifier and floc formation occurs in the installation. Small, light flocs with a settling velocity lower than the upflow velocity are transported with the water flow in an upward direction and collide with larger, heavier flocs. After attachment, the settling velocity increases until they reach the bottom of the

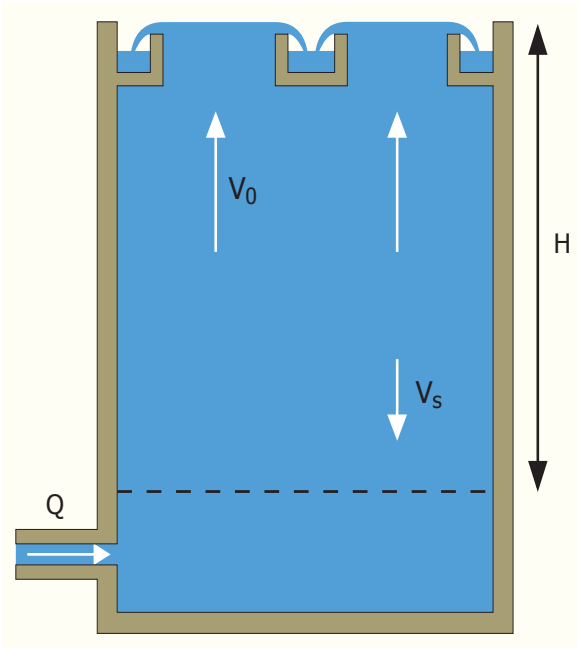



*Figure 26 - Vertical flow settling*

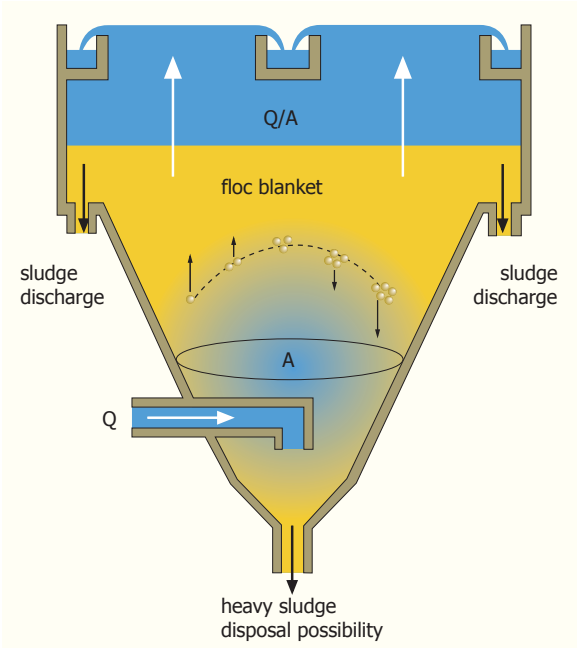



*Figure 27 - Floc blanket installation*

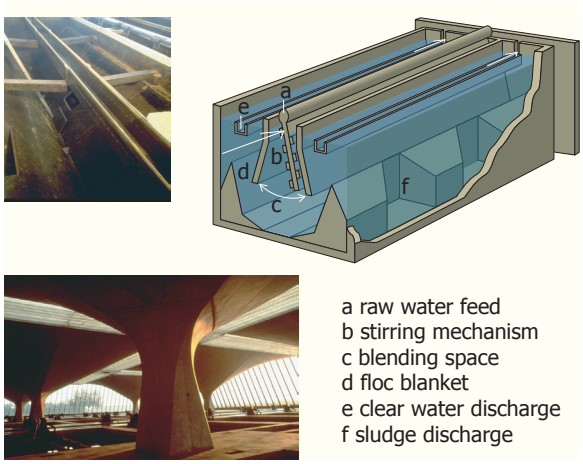



*Figure 28 - Floc blanket installation at Berenplaat*

tank, where a sludge removal device is installed.

Flocs that do not form aggregates are transported to the top of the installation where the surface area is the largest and the upflow velocity the lowest, and a floc blanket is formed.
The floc blanket is manipulated by a drain at the required thickness. The floc blanket has a filtering effect for even small flocs. Therefore, high efficiencies can be realized with relatively short residence times.

The plant at Berenplaat in Rotterdam is where one of the floc blanket clarifiers in the Netherlands is located (Figure 28). The inlet of the water is at the top of the installation. A stirring device creates turbulence in the floc formation chamber to increase the collision frequency. After leaving the mixing chamber, the flocs form a blanket and the effluent water is drained by troughs.

### 5.3 Tray settling tanks

The efficiency of discrete settling can be increased by applying horizontal baffles (false floors or trays) (Figure 29).
In section 2.2 it is shown that the settling efficiency for discrete particles is independent of the height of the settling tank. The application of horizontal baffles gives a double surface area and half of the surface loading, resulting in an increase in efficiency.
Horizontal baffles also improve the flow pattern.

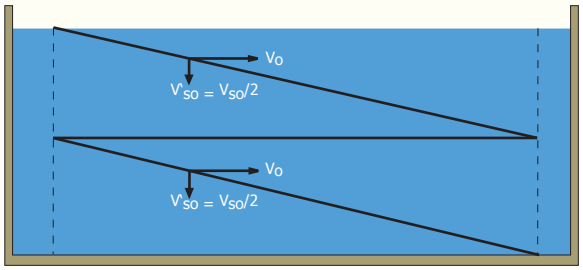



*Figure 29 - Tray settling*

The Reynolds number decreases and the Camp number increases. The flow becomes less turbulent and more stable.

### 5.4 Tilted plate settling

In tilted plate settling tanks, water passes baffles that are placed at a steep angle.

In Figure 30 an example is given of a counter-current tilted plate settling tank. The water flows in upward direction and the sludge settles on the plates and slides down. The angle of the plates must be about $55^o$ to $60^o$ to secure sludge removal.
In Figure 31 the flow in a counter-current tilted plate settling tank is pictured.
Using geometry the following surface loading can be derived:

$$s_o' = s_o \cdot \frac{w + t}{H \cdot \cos\alpha + w}$$

in which:

| | | | |
|---|---|---|---|
| $s_o'$ | = | vertical surface loading | [m/h] |
| w | = | distance between plates | [m] |
| t | = | thickness of plates | [m] |
| H | = | height of plates | [m] |
| α | = | angle of plates with the horizontal | [o] |

In a similar way the surface loading for a co-current flow can be derived:

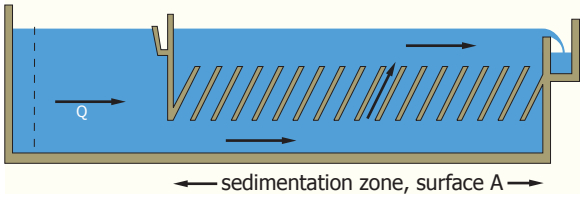



*Figure 30 - Counter-current tilted plate settling*

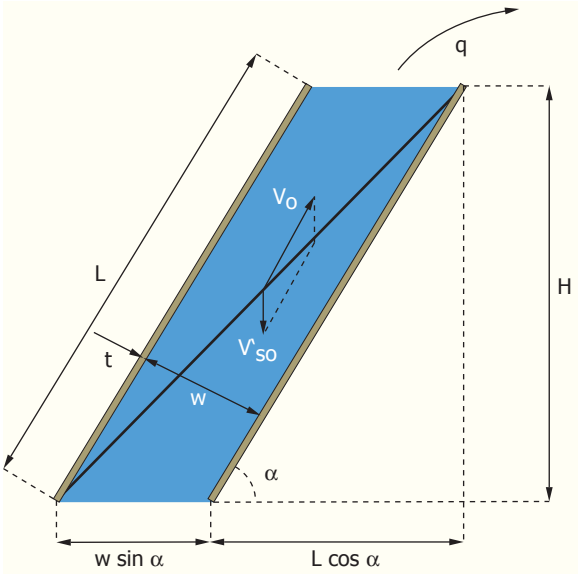



*Figure 31 - Flow through a tilted plate settler*

$$s_o' = s_o \cdot \frac{w + t}{H \cdot \cos\alpha - w}$$

In Table 3 the design parameters of Figure 31 that are applied in practice are given.
The angle of the plates in co-current systems can be gentler than in counter-current systems without deteriorating the sludge removal.
Substituting the values of Table 3 into the equations for surface loading for both co-current and counter-current systems results in:

$$s_o' \approx \frac{s_o}{20}$$

The space occupied by tilted plate settling tanks is thus a factor 20 smaller than is needed for horizontal flow tanks.

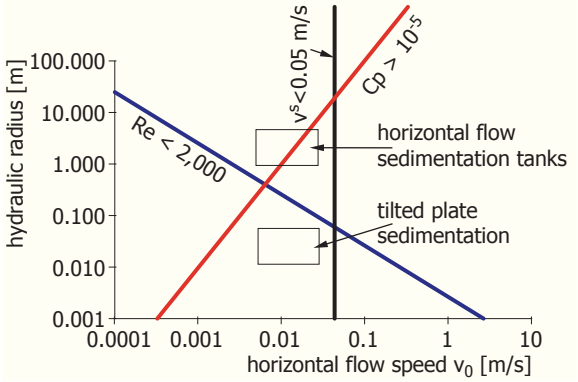



*Figure 32 - Hydraulic conditions for optimal settling*

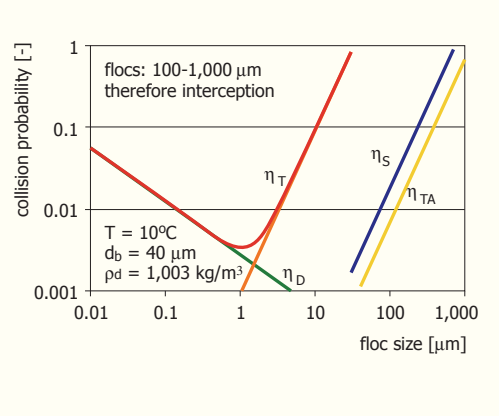
*Table 3 - Design parameters*

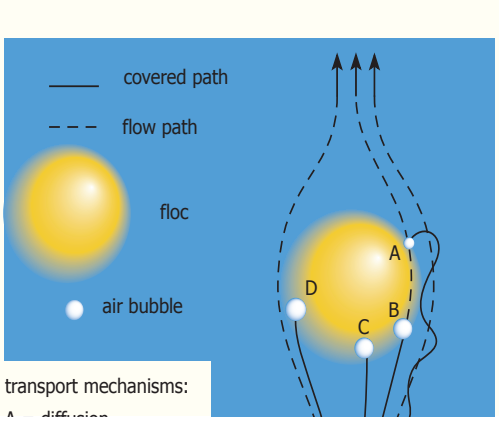
| Design parameters | Value |
|---|---|
| counter-current | $55^0$ - $60^0$ |
| co-current | $30^0$ - $40^0$ |
| H | 1 - 3 m |
| w | 3.4 - 8 cm |
| t | 5 mm |

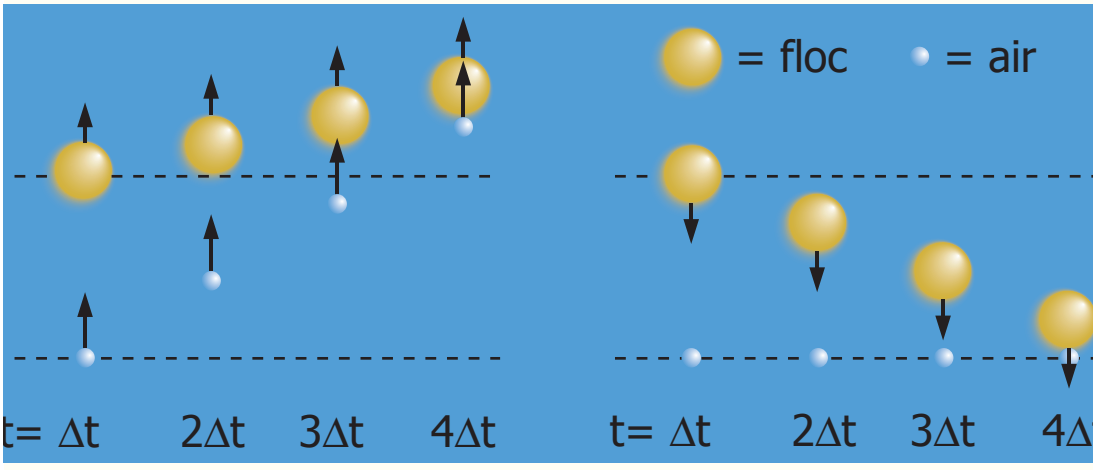
Both the Camp number and the Reynolds number depend on the hydraulic radius and the horizontal flow velocity.
In Figure 32 the stability boundary, $c_p > 10^{-5}$, and the turbulence boundary, Re < 2000, are given. In addition, the combinations of hydraulic radius and horizontal flow velocity of horizontal flow and tilted plate tanks applied in practice are shown.
From the graph it can be derived that the flow in horizontal flow tanks is turbulent and in some cases instable (and short-circuit flow can occur). The flow in tilted plate tanks, however, is favorable. The Reynolds number is always smaller than 2000, resulting in laminar flow; and the Camp number is always higher than $10^{-5}$, resulting in a stable flow without short-circuiting.

WATER TREATMENT

# Flotation



**Framework**
This module explains flotation

**Contents**
This module has the following contents:

1. Introduction
2. Principle
3. Theory
   3.1 Saturation unit for the supply of air
   3.2 Efficiency of the bubble filter in the filtration zone
   3.3 Collision probability between bubbles and flocs in the filtration zone
   3.4 Determination of the surface loading of the separation zone
4. Practice
   4.1 Design parameters
   4.2 Saturation
   4.3 Flotation tank

# 1 Introduction

In the past, flotation was mainly used in the Scandinavian countries and the UK.
Zevenbergen (Brabant Water) was the first Dutch treatment plant to make use of flotation as a floc removal process (1979). Other treatment plants where flotation is applied are Braakman (Evides), Elsbeekweg Enschede (Vitens) and Scheveningen (Dune Water Company South Holland).

Flotation is applied to remove flocs during surface water treatment and is preceded by floc formation (flocculation).
In the flotation process very small air bubbles are used to “air-lift” the flocs to the water surface.
The number and size of the air bubbles is the key factor for the upward velocity of the flocs, and thus for the separation efficiency of the process.
The upward velocity of the flocs is much higher than the sedimentation rate of these flocs. Therefore the surface load of flotation is also much higher, than the surface load for the sedimentation process.

Flotation is often selected as the floc-removal process for conditions which are less favourable for sedimentation:
- low temperature of the water, resulting in a reduced sedimentation rate due to increased viscosity
- high algae content in the water, resulting in a reduced sedimentation rate (algae might even float due to the release of oxygen during the night).

The air dosing level is an operation parameter which can be brought into compliance with varying operational conditions (temperature, viscosity, floc size , floc density).

# 2 Principle

The principle of flotation is shown in Figure 1.
The flocculated raw water is distributed at the bottom of the flotation tank and flows in an upward direction over the baffle.

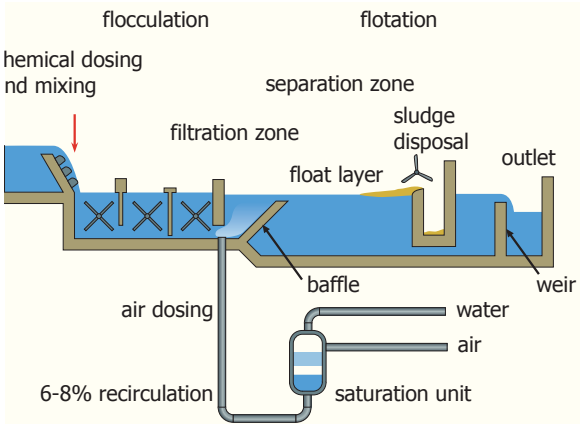



*Figure 1 - Principles of floc formation and flotation*

At the same time, a water flow with supersaturated air is supplied through nozzles. This water flow is called the saturation or recirculation flow. Due to the pressure drop in the supersaturated water at the nozzles, small air bubbles form.
The rising velocity of the air bubbles is higher than the water velocity, so the air bubbles collide with the flocs. Air pockets form beneath the flocs and the density of the aggregates decreases below the water density. As a consequence the aggregates will float on the water’s surface.

**Filtration zone**
Schematically the process in the zone before above mentioned baffle can be considered as a filtration process.
In Figure 2 a floc and an air bubble are represented in subsequent periods of time. It can be seen that the air bubbles rise faster than the flocs. Assuming the air bubbles are fixed (point of reference), the water moves with the flocs downwards and the flocs are filtered from the water by the air bubbles.

**Separation zone**
Removal of the floating flocs takes place in the

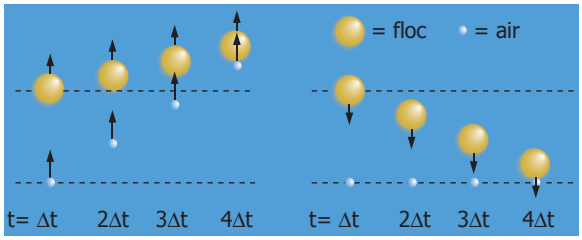



*Figure 2 - Filtration principle during flotation*

*Figure 3 - Float layer in separation zone*

separation zone.
The flocs and the air bubbles form a float layer together at the water's surface (Figure 3).
The float layer is transported by the water flow to a weir and is drained. To accelerate the removal process of the float layer, rotors can be placed at the sludge overflow.
The treated water flows under the sludge removal device and over a weir to the outlet of the flotation system (Figure 1).
In some installations the weir of the float layer removal device is flexible. When the weir is high, the float layer is thin and there is a risk that water will flow into the sludge removal system. When the float layer is thick, the risk is that air bubbles will escape from the float layer and the flocs will start to settle.

The height of the weirs (water and sludge) is of primary importance for the performance of the flotation system.

**Saturation unit**
The saturation water is made by the saturation unit (Figure 4).
The saturation unit is supplied by a water flow and an air flow.
The water flow is about 6% - 8% of the total water flow through the treatment plant and is abstracted downstream of the flotation process. The water is pumped into a pressure vessel, at a pressure of 4 to 8 bar.
Air is supplied into the pressure vessel via an air compressor. Because of the high pressure more air can be dissolved than is possible under atmos-

*Figure 4 - Saturation unit*

pheric circumstances.

The supersaturated water flow is transported from the saturation unit to the filtration zone and inserted

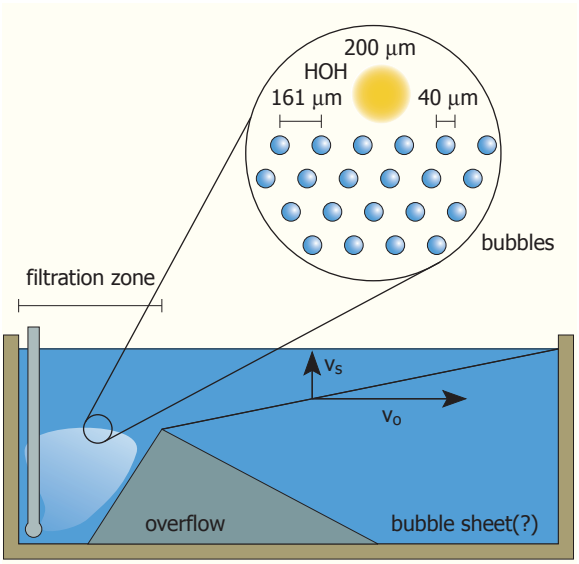



*Figure 5 - Size and distance between bubbles and flocs*

*Table 1 - Specific parameters of air bubbles and flocs*

| Air bubbles | | | Flocs | | |
|---|---|---|---|---|---|
| **parameter** | **value** | **unit** | **parameter** | **value** | **unit** |
| diameter | 10 - 100 | µm | diameter | 100 - 200 | µm |
| density | (1.5 - 3.0) • $10^{11}$ | bubbles/$m^3$ | floc density | (2.5 - 19) • $10^7$ | flocs/$m^3$ |
| distance between bubbles | 150 - 188 | µm | distance between flocs | 3600 - 3700 | µm |
| air dosage | 5 - 10 | l/$m^3$ | density | 1003 - 1006 | kg/$m^3$ |
| | | | particle concentration | 10 -25 | g/$m^3$ |

through the distribution nozzles.

**Specific parameters**

To give an idea of the order of magnitude of the air bubbles and flocs, some values that are encountered in practice are represented in Table 1.

Depending on the type of nozzle, the median diameter of an air bubble lies between 30 and 40 µm. In Figure 5 some specific parameters determined at an air dose of 8 l/$m^3$ are represented.
From Figure 5 and Table 1 it can be concluded that collision between air bubble and floc is inevitable, because the distance between two air bubbles is smaller than the diameter of a floc.

## 3 Theory

### *3.1 Saturation unit for the supply of air*

The amount of air that can be dissolved in a certain volume of water depends on the pressure and the water temperature and can be calculated with Henry's Law:

$$c_s = k_H \cdot c_a = k_H \cdot \frac{MW \cdot p}{R \cdot T}$$

in which:

$c_s$ = saturation concentration of gas in water [g/$m^3$]
$k_H$ = distribution coefficient [-]
$c_a$ = concentration of gas in air [g/$m^3$]
MW = molecular weight of gas [g/mol]
p =partial pressure of gas in air [Pa]
R =universal gas constant =8.3142 [J/(°K•mol)]
T =(air) temperature [K]

When the concentration of gas in water is calculated with Henry's Law, the total air pressure can be used instead of the partial pressure. Then, the concentration ca represents the specific density of air at the prevailing temperature and pressure.

In Table 2 the distribution coefficient for different air temperatures is represented.
By increasing the pressure, the amount of gas that can be dissolved in a volume of water increases proportionally, as is shown in Figure 6, where the saturation concentration is given as a function of pressure (atmospheric pressure at sea level is 101,325 kPa).

The gas exchange between water and air is more extensively explained in the module on aeration and gas stripping.

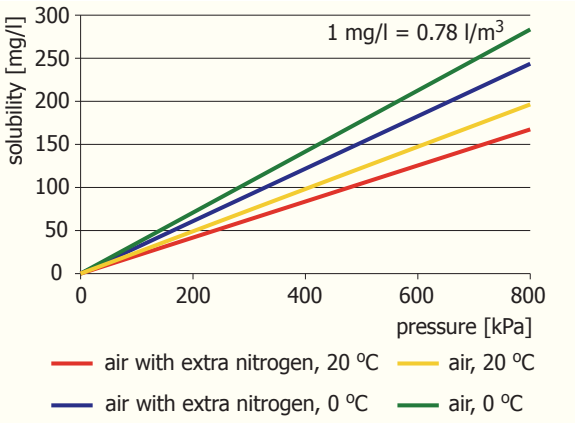



*Figure 6 - Solubility of air in water*

*Table 2 - $k_H$-values at different temperatures and the molecular weight of air*

| Concentration of gas in air | T = 0°C | T = 10°C | T = 20°C | T = 30°C | MW [g/mol] |
|---|---|---|---|---|---|
| 79% $N_2$, 21% $O_2$ | 0.0288 | 0.0234 | 0.0200 | 0.0179 | 28.84 |

### 3.2 *Efficiency of the bubble filter in the filtration zone*

The efficiency of a bubble filter in the filtration zone is equal to the proportion of flocs that collide with the bubble filter.
This portion of flocs can be derived from the mass balance, assuming a permanent attachment with one or more air bubbles.

**Kinetics equation**

In Figure 7 a plug flow is represented together with the principle of collision between flocs and air bubbles (see also Figure 2).
From the mass balance for a unit element dH (Figure 7), the kinetic equation for the collision of flocs and air bubbles in a plug flow can be derived.
The mass balance for a unit element dH is:

$$Q \cdot N_d dt = Q \cdot \left( N_d + \left( \frac{dN_d}{dH} \right) dH \right) dt +$$

$$\alpha_{db} \cdot \eta_T \cdot N_d \cdot N_b \cdot dV \cdot A_b dH$$

in which:

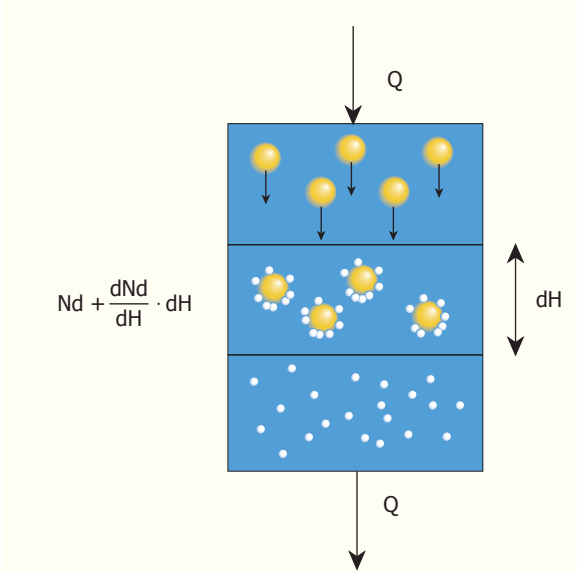



*Figure 7 - Mass balance during filtration of the flocs*

$N_d$ = floc density [flocs/$m^3$]
$\alpha_{db}$ = collision coefficient between air bubble and floc [-]
$\eta_T$ = collision frequency between air bubble and floc [-]
$N_b$ = air bubble density [air bubbles/$m^3$]
dV = volume of unit element dH [$m^3$]
$A_b$ = projected area of an air bubble [$m^2$/air bubble]

The second part of the right half of the equation represents the number of flocs that collides and attaches to an air bubble in the unit element dH.
The number of flocs depends on the collision frequency, the collision efficiency, the number of flocs and bubbles in the unit element and the size of the projected collision area of the bubble.

The collision efficiency is determined by pre-treatment of the water and is negatively influenced by turbulence in the filtration zone. The collision frequency is elaborated on in paragraph 3.3.
Rearranging the mass balance leads to:

$$\frac{dN_d}{dH} = -\alpha_{db} \cdot \eta_T \cdot N_d \cdot N_b \cdot Ab$$

with

$$dH = v_{db} \cdot dt$$

in which:
$v_{db}$ = approaching velocity between air bubble and floc [m/s]

The kinetics equation for collision between air bubbles and flocs becomes:

$$\frac{dN_d}{dt} = -\alpha_{db} \cdot \eta_T \cdot N_d \cdot N_b \cdot A_b \cdot v_{(}$$

Because the air bubbles rises much faster than the flocs, it is assumed that the approaching velocity

is equal to the bubbles' rising velocity.
The rising velocity of an air bubble under laminar flow conditions can be calculated with Stokes' law:

$$v_b = \frac{1}{18} \cdot \frac{g \cdot d_b^2}{\nu}$$

in which:
$v_b$ = rising velocity of air bubbles [m/s]
$d_b$ = diameter of air bubbles [m]
$\nu$ = kinematic viscosity [$m^2/s$]

The air bubble density is equal to the air dosage divided by the volume of the air bubble:

$$N_b = \frac{\varphi_b}{\frac{1}{6} \cdot \pi \cdot d_b^3}$$

in which:
$\varphi_b$ = air dosage [$m^3$ air/$m^3$ water]

and $A_b$ is:

$$A_b = \frac{1}{4} \cdot \pi \cdot d_b^2$$

Substituting $v_{db}$ with $v_b$ and inserting $N_b$ and Ab gives the following kinetics equation:

$$\frac{dN_d}{dt} = -\frac{1}{12} \cdot \frac{\alpha_{db} \cdot \eta_T \cdot d_b \cdot \varphi_b \cdot g}{\nu} \cdot N_d$$

This equation is a first-order reaction and is equivalent to dc/dt = -kc in filtration and activated carbon filtration.

**Efficiency**
Under the assumption that the filtration zone can be schematized by a plug flow, the equation mentioned above can be integrated with the following boundary conditions:
- at t = 0: $N_d = N_{d,i}$
- at t = t : $N_d = N_{d,e}$

in which:
$N_{d,i}$ = floc density of the influent [flocs/$m^3$]
$N_{d,e}$ = floc density of the effluent [flocs/m3]

After integration, the efficiency of filtration can be expressed as:

$$R = \frac{N_{d,i} - N_{d,e}}{N_{d,i}} = 1 - e^{\left(-\frac{1}{12} \cdot \frac{\alpha_{db} \cdot \eta_T \cdot d_b \cdot \varphi_b \cdot g \cdot \tau}{\nu}\right)}$$

in which:
R = efficiency of a bubble filter [-]
$\tau$ = residence time in the filtration zone [s]

### 3.3 Collision probability between bubbles and flocs in the filtration zone

In the equation of the efficiency of filtration, the collision probability between air bubbles and flocs is incorporated.
The collision probability is the ratio between the actual number of collisions and the possible number of collisions between air bubbles and flocs.
It is assumed that the floc is situated in a water column above the air bubble with a surface area (per-pendicular to the rising direction) equal to the projected area of the air bubble.

In an analogy of filtration, four different transport mechanisms can be described for the collision between air bubble and floc:
- diffusion $\eta_D$, collision caused by Brownian motion of mainly small flocs and particles
- interception $\eta_I$, collision because the floc trajectory approaches the air bubble and interception of the floc by the air bubble is possible
- sedimentation $\eta_S$, collision caused by large and heavy flocs that deviate from the original trajectory
- inertia $\eta_{TA}$, collision caused by mainly large and heavy flocs that deviate from the curvature of the original trajectory.

For these transport mechanisms (Figure 8), the collision probability η or the Single-Collector Collision Efficiency (SCCE) can be quantified:

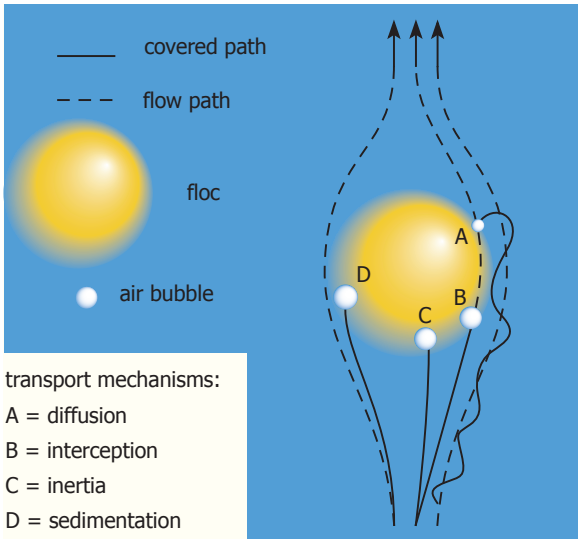



*Figure 8 - Transport mechanisms*

$$\eta_D = 6{,}18 \cdot \left(\frac{k_b \cdot T}{g \cdot \rho_w}\right)^{\frac{2}{3}} \cdot \left(\frac{1}{d_d}\right)^{\frac{2}{3}} \cdot \left(\frac{1}{d_b}\right)^2$$

$$\eta_I = \frac{3}{2} \cdot \left(\frac{d_d}{d_b}\right)^2$$

$$\eta_S = \left(\frac{\rho_d - \rho_w}{\rho_w}\right) \cdot \left(\frac{d_d}{d_b}\right)^2$$

$$\eta_{TA} = \frac{g \cdot \rho_d \cdot d_b \cdot {d_d}^2}{324 \cdot v^2 \cdot \rho_w}$$

in which:

$k_b$ = Boltzmann constant = $1.38 \cdot 10^{-23}$ [J/°K]
T = absolute water temperature [K]
$d_d$ = diameter of the flocs [m]

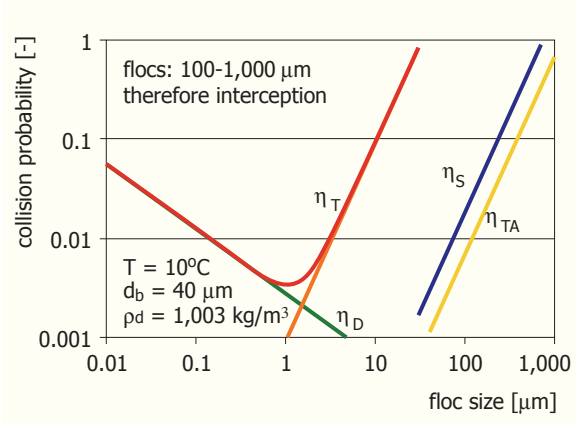



*Figure 9 - Collision probability*

$\rho_w$ = density of water [kg/m$^3$]
$\rho_d$ = density of flocs [kg/m$^3$]

The total collision probability $\eta_T$ is equal to the sum of the separate collision probabilities of the individual transport mechanisms:

$$\eta_T = \eta_D + \eta_I + \eta_S + \eta_T$$

In Figure 9 the collision probabilities are represented as a function of floc diameter and the following can therefore be concluded:

- the diffusion mechanism is predominant for flocs smaller than 1 µm
- the interception mechanism is predominant for flocs larger than 5 µm.

From the equation of collision probability of interception and Figure 9, it can be concluded that with an air bubble of diameter 40 µm the collision probability $\eta_T$ =1, if the floc diameter is larger than 32 µm. After preceding floc formation the floc size is between 100 and 1000 µm.
Consequently, in practice, the interception mechanism is predominant and the collision probability is equal to 1.

The progress of the total collision probability in Figure 9 is equivalent to the curves of the collision probability in filtration and floc formation theory.
Similar to filtration, the collision probability is minimal for particles size of about 1 µm.
The main difference between the processes, however, is that the collision probability in flotation processes is an order of magnitude larger than in filtration and, therefore, the flocs will collide more easily during flotation.

### *3.4 Determination of the surface loading of the separation zone*

Removal of air bubble-floc aggregates takes place in the separation zone.
All aggregates are removed if the time needed for the flotation of an aggregate at the bottom of the separation zone is less than the residence time in the separation zone (the opposite of discrete

settling):

$$\frac{t_b}{t_{st}} > 1$$

in which:
$t_b$ = residence time in the separation zone [s]
$t_{st}$ = time an aggregate needs to reach the water surface [ s]

Assuming a plug flow in the separation zone:

$$v_{st} > q \cdot \frac{1}{1-m}$$

in which:
$v_{st}$ = rising velocity of the air bubble-floc aggregate [m/s]
q = surface loading of the separation zone [$m^3/(m^2 \cdot s)$]
m = fraction dead space (eddies) in the separation zone [-]

It can be concluded that the maximum surface loading is determined by the rising velocity of the aggregates in the separation zone.

**Rising velocity of the air bubble-floc aggregates**

Assuming a laminar flow and spherical aggregates, the rising velocity can be calculated with Stokes' law:

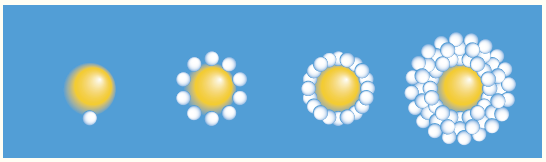
$$v_{st} = \frac{1}{18} \cdot \frac{g}{v} \cdot \left( \frac{\rho_w - \rho_a}{\rho_w} \right) \cdot d_a{}^2$$

in which:
$\rho_a$ = density of the aggregate [$kg/m^3$]
$d_a$ = diameter of the aggregate [m]

The density and the diameter of the aggregate can be determined with the following equation:

$$\rho_a = \left( \frac{\beta}{\beta+1} \right) \cdot \rho_b + \left( \frac{1}{\beta+1} \right) \cdot \rho_d$$

diameter air bubbles = 40 µm
diameter floc = 200 µm



| | | | | |
|---|---|---|---|---|
| β | 0.008 | 0.08 | 0.8 | 8.0 |
| $\frac{d_a}{d_d}$ | 1.00 | 1.03 | 1.21 | 2.08 |
| $\rho_a$ | 995.1 | 928.8 | 557.8 | 112.5 kg/m³ |
| $v_{st}$ | 0.43 | 6 | 44.7 | 152 m/s |

*Figure 10 - Air bubble-floc aggregate parameters*

$$d_a = \sqrt[3]{1+\beta} \cdot d_d$$

in which:
β = volume ratio between air bubbles and flocs in the aggregate [-]

In Figure 10 the influence of the volume ratio on the diameter, density and rising velocity of the aggregate is represented.

In calculating the diameter of the aggregate da the volume of the air bubbles is assumed to be divided over the entire surface of the floc. It is practically physically impossible for more than one bubble layer to exist. For a floc diameter of 200 µm, this results in a maximum volume ratio of 1.
The volume ratio between the total volumes of inserted air and flocs is about 500. Thus, only 0.2% of the total inserted air is effectively used during the floc removal process.
The applied (high) air dosage, however, is necessary for a sufficiently high efficiency of the bubble

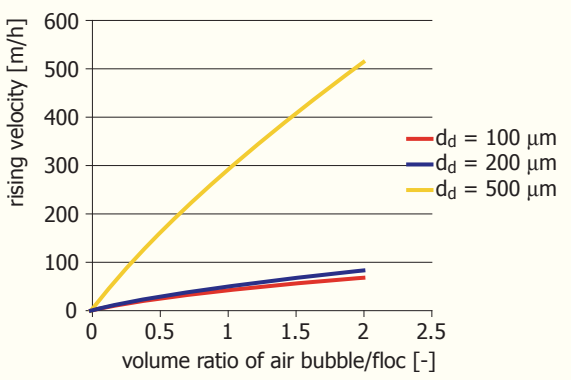



*Figure 11 - Rising velocity for different β*

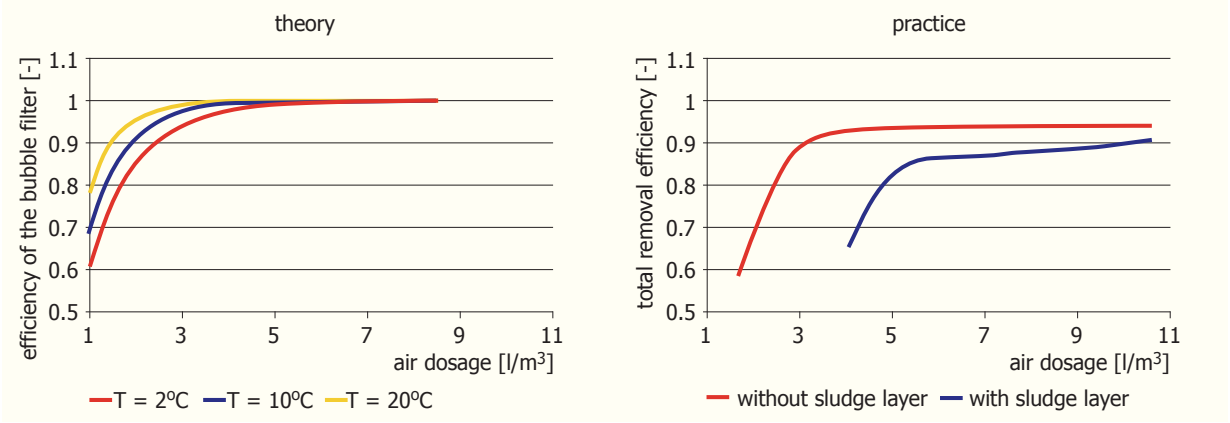



*Figure 12 - Influence of air dosage in theory and practice (total efficiency)*

filter in the filtration zone.

In Figure 11 the rising velocity is represented as a function of volume ratio for flocs with a diameter of 100 µm, 200 µm and 500 µm.
Because the volume ratio between air bubbles and flocs is (physically) restricted (bmax » 1), an increase in the rising velocity vst can only be realized by an increase in floc size.

## 4 Practice

### *4.1 Process parameters*

**Air dosage**

The relationship between air dosage and efficiency of the bubble filter is exponential, as seen in Figure 12.
In addition it can be concluded that at lower water temperatures, higher air dosages are required to obtain the same efficiency.
The air dosage is an operation parameter for the efficiency of the bubble filter.

In Figure 12 the total removal efficiency, measured in practice, is represented as a function of the air dosage. The total efficiency consists of the efficiency in the filtration zone and the efficiency in the separation zone.
The total efficiency approaches the value 0.9 – 0.95 and not 1.
This can be due to the occurrence of short circuit flows in the filtration zone, resulting in a decrease in the efficiency in the filtration zone or to the limited influence of the air dosage on the efficiency in the separation zone (less than 1% of the air is effectively used for separation), and separation in practice is not optimal.

**Contact time**

Theoretically the contact time should be longer

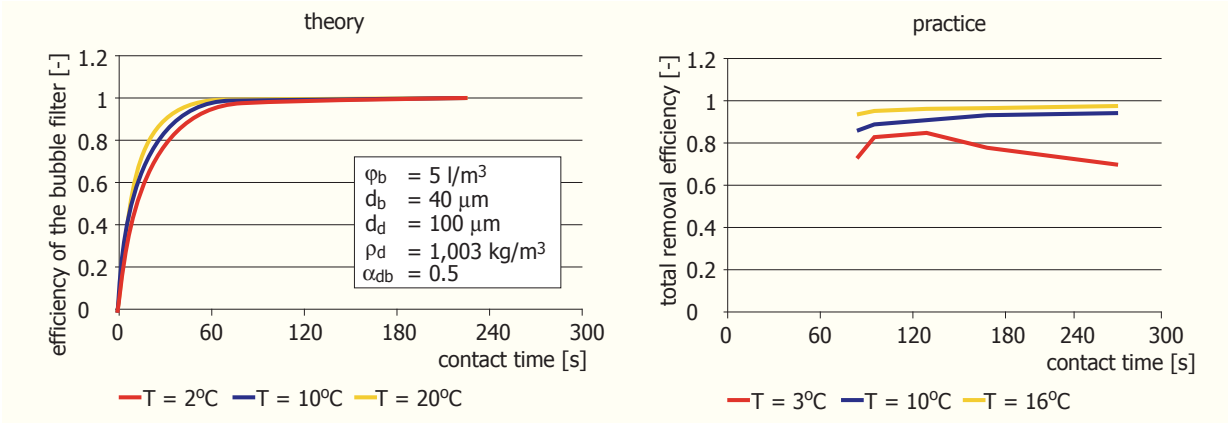



*Figure 13 - Influence of contact time in theory and practice (total efficiency)*

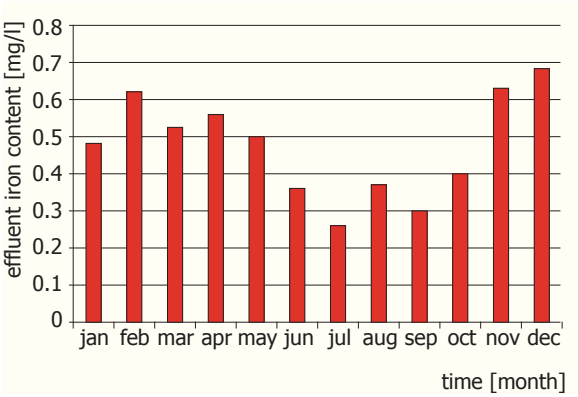



*Figure 14 - Seasonal influence by temperature variation*

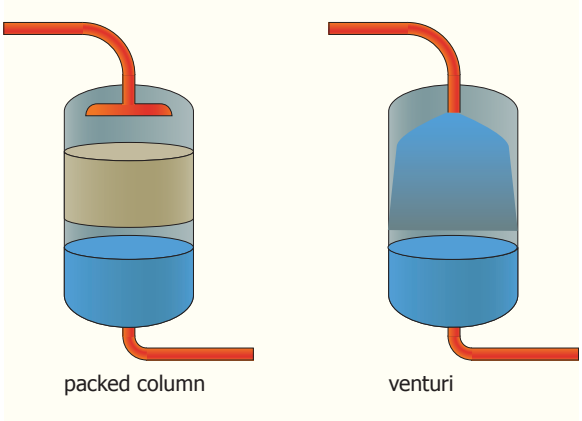



*Figure 15 - Saturation units*

than 90 seconds, as seen in Figure 13.
In practice contact times of 54 to 126 seconds are applied.

In this figure, the results of measurements on full-scale plants, where the contact times are varied, are represented.
In general, the theory is confirmed, but an efficiency of 1 is, even with infinite contact times, not reached.
The reason is that in cases where the contact times are increased, the water flow must be decreased (for the same flotation tank). At low water flows, plug flow no longer occurs and the efficiency will be lower than 1.

**Temperature**
The water temperature determines the viscosity of the water, influencing both filtration and separation.
From calculations it can be derived that the air dosage at 5 °C must be a factor 1.6 to 1.7 higher than at 20 °C to obtain the same efficiency (Figure 12).

It can also be derived that the rising velocity of an aggregate in the separation zone is 1.6 times lower at 2 °C than at 20 °C.
Therefore, the efficiency in the separation zone will be lower at lower temperatures.
In Figure 14 the progress of the residual iron concentration in a flotation system is represented. The temperature effect is obvious.

## *4.2 Saturation*

**Saturation unit**
In practice two types of saturation units are applied.
The saturation units that make use of a packed column, in analogy of tower aerator and vacuum gas transfer system, and the venturi aeration units.
Both systems are schematically represented in Figure 15.

For the design and functioning of the saturation units, reference is made to the module on aeration and gas stripping.

**Nozzles**
In Figure 16 the principle of a nozzle that is applied in the filtration zone for the distribution of saturation water is shown.

In practice Bete, AKA and WRC nozzles are applied.
In Figure 17 the median bubble diameter for the

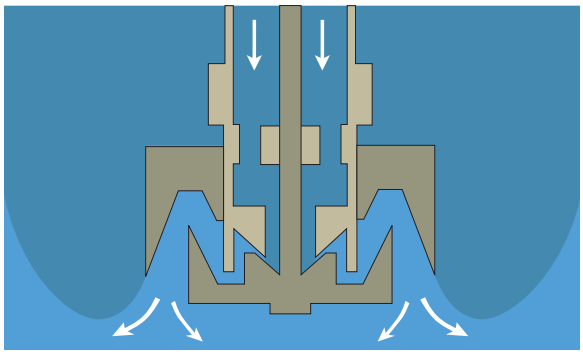


*Figure 16 - Nozzle*

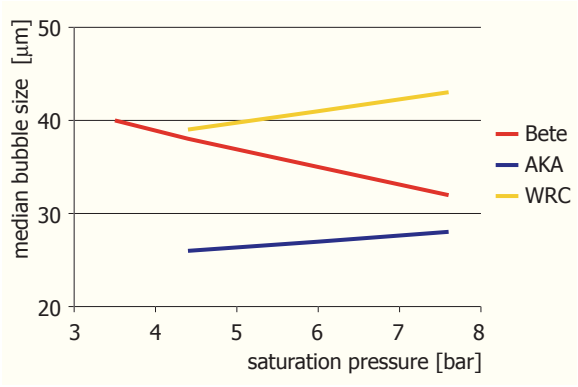



*Figure 17 - Performance of three different nozzles*

different nozzles is given as a function of saturation pressure of the recirculation flow.

Based on the required median bubble diameter, a nozzle and a saturation pressure can be chosen. The median bubble diameter is for most of the nozzles between 30 µm and 40 µm.

### 4.3 Flotation tank

In Figure 18 a flotation tank with filtration or contact zone, separation zone and baffle is represented. These will be discussed in the following paragraphs.

**Filtration zone**

The filtration zone ($V_c$ in Figure 18) is designed based on a contact time longer than 90 seconds. To obtain a plug flow in the filtration zone, the length/width ratio must be higher than 5 (long and narrow).
A column reactor would thus be suitable for the filtration zone.

In existing flotation tanks, the saturation water is released into the filtration zone, resulting in local

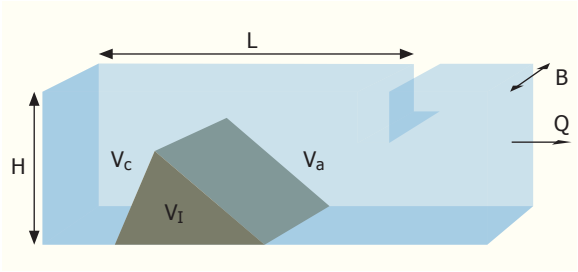



*Figure 18 - Design of flotation tank*

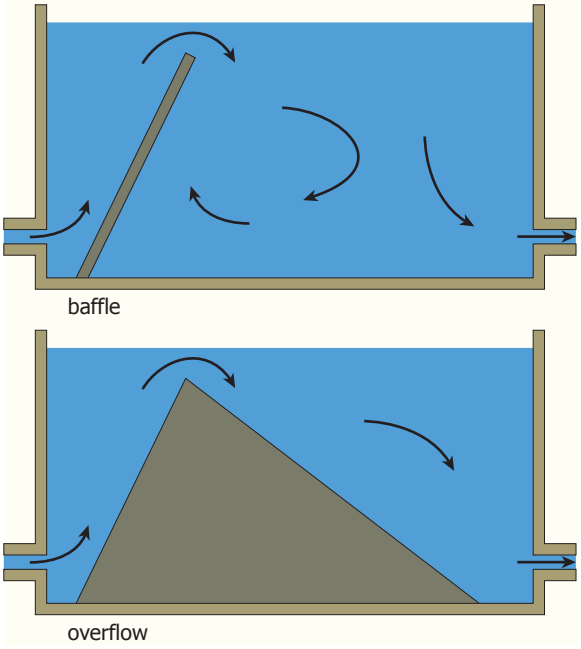



*Figure 19 - Division of filtration and separation zone*

(near the nozzles) velocity gradients of 20-30 m/s, turbulence and break-up of flocs and bubble-floc aggregates.
Therefore, strong (and thus small) flocs must be formed during floc formation to resist the turbulent flows.

Moreover, this turbulence can be minimized by the arrangement of the nozzles. An example of this is the application of a separate nozzle zone outside the main water flow.

**Separation zone**

The separation zone ($V_a$ in Figure 18) is designed based on surface loading, which is determined from the rising velocity of the bubble-floc aggregates and takes the dead zones Vl into account.
In practice, surface loadings of 10 to 25 $m^3/(m^2 \cdot h)$ are applied.
The residence times in the separation zone vary between 5 and 10 minutes.
The height H must be about 2 meters to avoid large flow gradients and the zone must be long and narrow to approach a plug flow.
Finally, the water flow must be uniformly distributed and collected over the width of the separation zone to limit the fraction of dead zone.

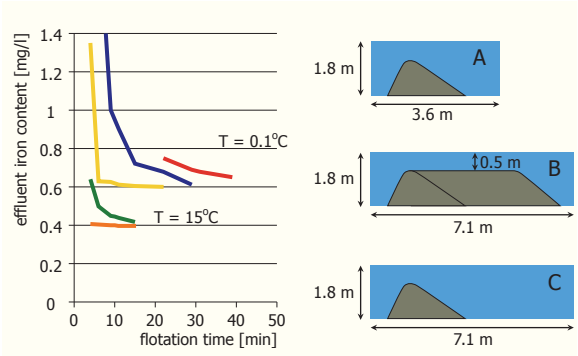



*Figure 20 - Design of flotation tank with different overflow structures*

**Division between filtration zone and separation zone**

The division between the filtration zone and separation zone is achieved with a baffle or an overflow (Figure 19).

Comparing the overflow structure with the baffle, no difference is observed in effluent quality and residence time. In both cases the residence time is about 70 % of the gross residence time (Figure 18):

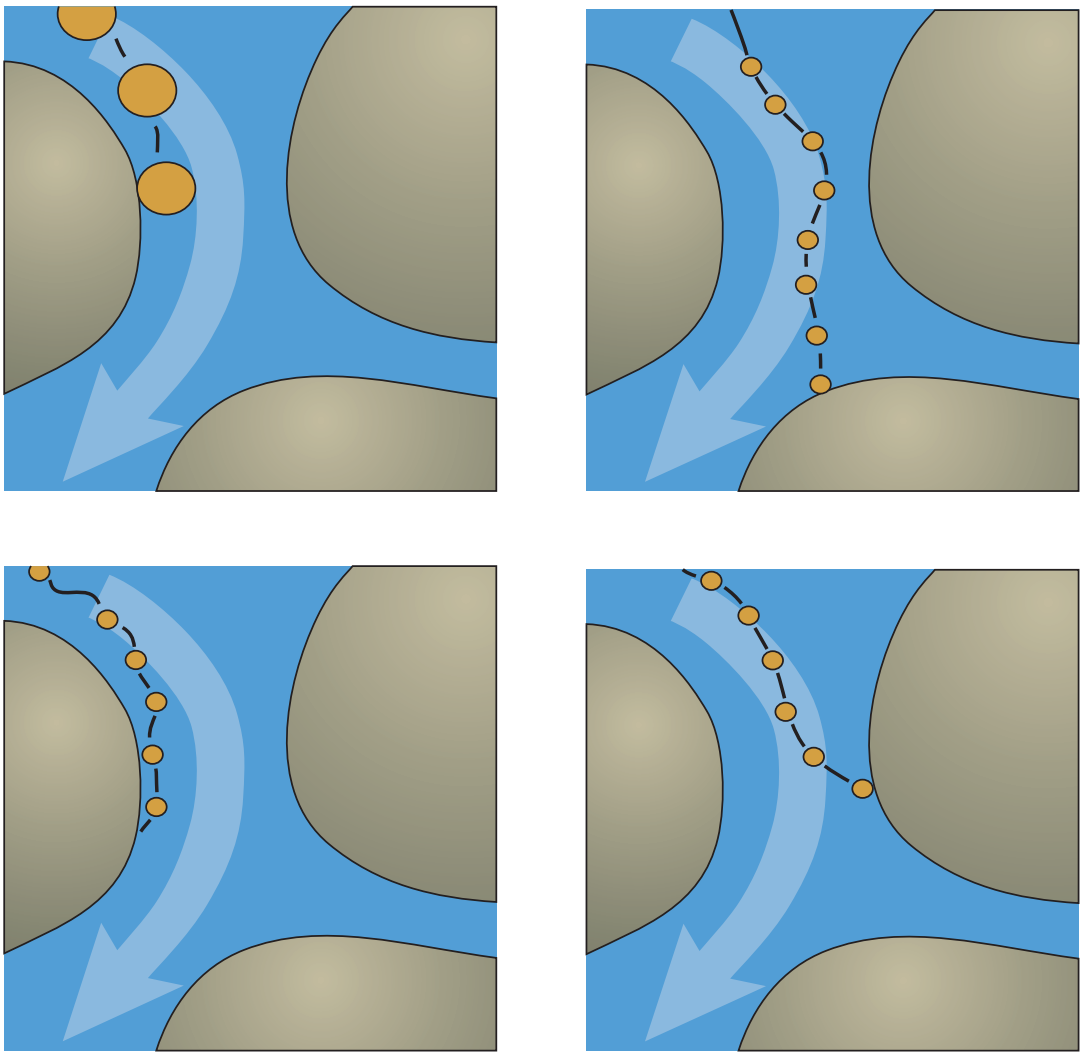
$$t_{bruto} = \frac{Q}{V_{br}} = \frac{Q}{L \cdot B \cdot H} = \frac{Q}{V_c} + \frac{Q}{V_a} + \frac{Q}{V_l}$$

The flow conditions with an overflow are favorable and less sludge is deposited. It can therefore be concluded that an overflow is preferred above the baffle.

Different types of overflow structures are represented in Figure 20.

WATER TREATMENT

# Filtration



**Framework**
This module explains filtration

**Contents**
This module has the following contents:

1. Introduction
2 Principle
  2.1 Filtration mechanismsna
  2.2 Filtration, column tests
3 Theory
  3.1 Filtration
  3.2 Backwashing
4 Practice
  4.1 Backwashing
  4.2 Filter bottom
  4.3 Filter material
  4.4 Filter troubles
5 Alternative applications of filtration
  5.1 Multiple layer filtration
  5.2 Pressure filtration
  5.3 Upflow filtration
  5.4 Limestone filtration
  5.5 Continuous filtration
  5.6 Dry filtration
  5.7 Slow sand filtration

# 1 Introduction

In general, filtration is a process where water flows to a permeable layer, either a membrane, filter paper, a sieve, a porous medium or such.
In water treatment, filtration is a process where water flows through granular material (often sand) while suspended solids (sand, clay, iron and aluminum flocs) are retained, substances are biochemically decomposed and pathogenic microorganisms (bacteria, viruses, protozoa) are removed.

The suspended solids slowly fill the pores, resulting in an increase in hydraulic resistance.
The suspended solids are removed by periodically cleaning the filter beds. This prevents the resistance from becoming too high or breaking through of suspended solids to occur.
Filters are also used as a chemical and biological reactor. This is mainly of importance for the treatment of groundwater where the oxidation of iron, manganese, ammonium and, in case of poor gas stripping, methane takes place.
The removal of pathogenic microorganisms is of importance for surface water treatment, and the efficiency is approximately 90 to 99%. The removal of pathogenic microorganisms occurs by decay and retention on the (sand) grains.

The most common application of filtration is the rapid sand filtration (Figure 1).
Rapid sand filtration consists of a bed with a coarse granular medium (0.8-1.2 mm) and supernatant water. The filtration velocities (between 5 and 20 m/h) are controlled by varying the supernatant water level (inlet-controlled) or by operating a valve at the outlet pipe (outlet-controlled).

Due to clogging, maximum resistance is reached, and the filter bed must be cleaned by backwashing. During backwashing the filter bed is expanded, and the accumulated suspended solids are removed. The backwash water is drained through a central trough to a waste receptacle. The backwash frequency is repeated every few days.

Rapid sand filters are present in nearly every water treatment plant.
Surface water treatment uses the filters after floc formation and removal to get rid of the remaining flocs and pathogens and to decompose ammonium.
In groundwater treatment, the filters are usually placed after aeration to remove iron flocs, manganese and ammonium. With softening, filters are often placed after pellet reactors to remove the ‘carry-over’.

# 2 Principle

## *2.1 Filtration mechanisms*

When water flows through the filter bed, suspended and colloidal particles are retained by the filter material.
Particles that are larger than the pores in the filter bed will remain on the bed (Figure 2). With rapid filtration this does not occur often, because the larger particles (iron or aluminum flocs) are already removed in the preceding floc removal process (sedimentation or flotation).
If smaller filter material is used, the pores are also smaller and the screening process results in the so-called cake filtration. The cake will also retain small particles, and treatment occurs mainly in the

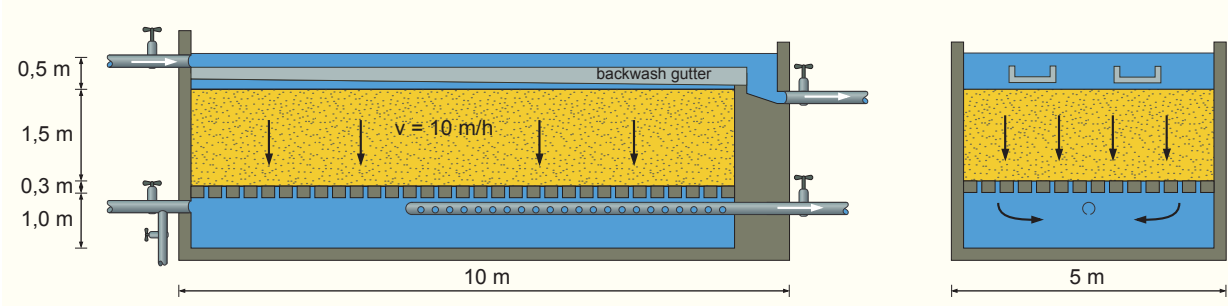



*Figure 1 - Principle of rapid sand filtration*

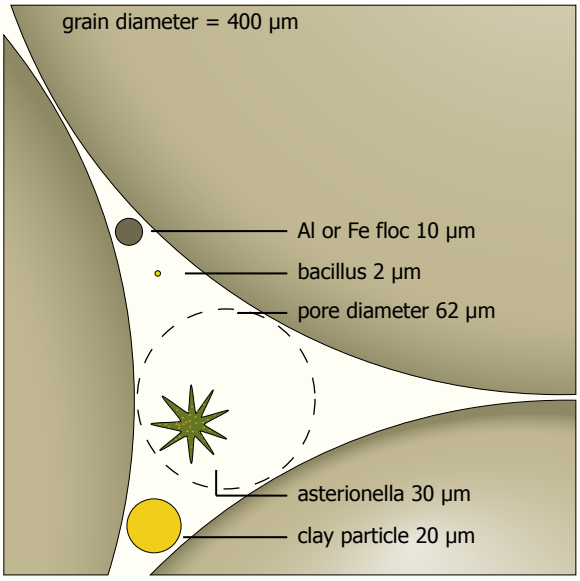



*Figure 2 - Principle of screening*

top layer of the filter.
The disadvantage of cake filtration is that with high concentrations of suspended and colloidal particles rapid clogging of the filter bed occurs.

During rapid filtration the removal of suspended and colloidal particles usually occurs inside the filter bed.
The clogging will thus be spread over the entire height of the filter bed.
The suspended and colloidal particles are transported to the filter material in different ways (Figure 3).
Generally, the particle follows the trajectory of the water that flows through the filter bed. This trajectory follows the complicated pore structure of the bed. When the trajectory curves, a heavy particle can be transported to the filter material due to inertia. In case the trajectory approaches the filter grains, particles can also be intercepted. Heavy particles are especially subject to sedimentation, lighter particles to diffusion. Due to these mechanisms, the particle can switch to other trajectories that flow nearer to a grain or can collide directly with a grain, and remains at the surface or the grain.

When the suspended and colloidal particles collide on the filter grains, attachment could take place.
There are two types of forces that result in attraction and repulsion of the particles.
The VanderWaals forces ensure that two bodies are attracted.
Electrostatic forces can have an attracting or repulsing effect, depending on the charge of the particles.
In general the filter material (sand) and the suspended and colloidal particles have a negative charge and repulsion takes place.

Attachment of the particles depends on the magnitude of both opposing forces. If the particles are destabilized by addition of trivalent iron or aluminum salts attachment will be easier than without destabilization.

In addition to physical processes to remove suspended and colloidal solids as described above, chemical and biological processes occur in the filter bed.
From groundwater iron(II) and manganese must be removed by oxidation. By addition of oxygen in the preceding aeration step iron(II) will be transformed into iron(III) and iron flocs are formed. The iron flocs are removed by the same mechanisms as described for the removal of suspended and colloidal particles.

Manganese is transformed to manganese oxide in the presence of previously deposited manganese oxide (catalytic process).

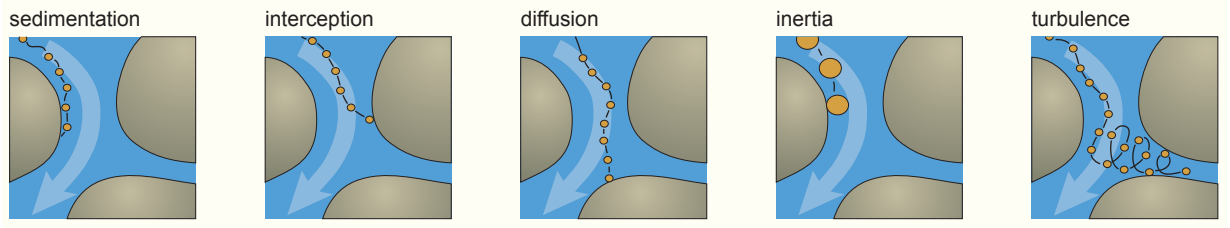



*Figure 3 - Transport of impurities towards the grain*

**Chemical and biological decompositions in the filter bed**

$4\ Fe^{2+} + O_2 + 2\ H_20 + 8\ HCO_3^- \rightarrow 4\ Fe(OH)_3 + 8\ CO_2$
$2\ Mn^{2+} + O_2 + 6\ H_2O \rightarrow 2\ MnO_2 + 4\ H_3O^+$
$NH_4^+ + 2\ O_2 + 2\ H_3O^+ \rightarrow NO_3^- + 3\ H_2O + 2\ CO_2$

Consequently, it can take several months before the manganese removal in the filter bed is initialized. Therefore, measures have to be taken to avoid total removal of manganese oxide during backwashing to keep the oxidation process alive.
There are also indications that manganese is oxidized in the presence of bacteria, suggesting a biological process.
Other biological processes in the filter bed are the decomposition of methane, ammonium, and (biodegradable) organic matter.
The decomposition of methane in the filter bed has to be avoided, because it results in an uninhibited growth of bacteria, which can lead to clogging and breakthrough. Methane, therefore, must be removed early in the process.

Ammonium is transformed into nitrate in two steps.
A group of nitrifiers (e.g., nitrosomonas) take care of the transformation of ammonium into nitrite, another group of nitrifiers (e.g., nitrobacteria) transform nitrite into nitrate.
The nitrifiers are situated on the surface of the filter material and for their growth they use energy that is produced during the transformation of ammonium or nitrite. The amount of ammonium that can be transformed depends on the growth rate of the bacteria, the size of the bacteria population, and the amount of ammonium that is transported to the bacteria (by diffusion).
In the beginning the growth rate of the bacteria is optimal and is not inhibited. The population during this lag phase is small and little ammonium is transformed. After some time the population will grow and finally stabilize (growth is equal to decay). At that time the maximum ammonium removal occurs.

For iron and manganese removal the oxygen consumption is limited.
Iron is normally present in concentrations lower than 10 mg/l and manganese concentrations are seldom above 1 mg/l. The oxygen concentrations that are needed for these reactions are, after aeration, dissolved in water (approximately 10 mg/l).
For biological processes the oxygen consumption is considerably higher. When low concentrations of ammonium and/or methane are present in the water (few mg/l), the amount of oxygen that can be dissolved in water under atmospheric conditions is insufficient to complete these reactions. A single filtration step is no longer sufficient. Furthermore, high bacteria concentrations in the filter can increase the risk for the growth of Aeromonas.

## 2.2 *Filtration, column tests*

It is important to learn from the experiences of other treatment plants that have to deal with similar water to determine the dimensions of a filter.
Further information can be obtained with a test

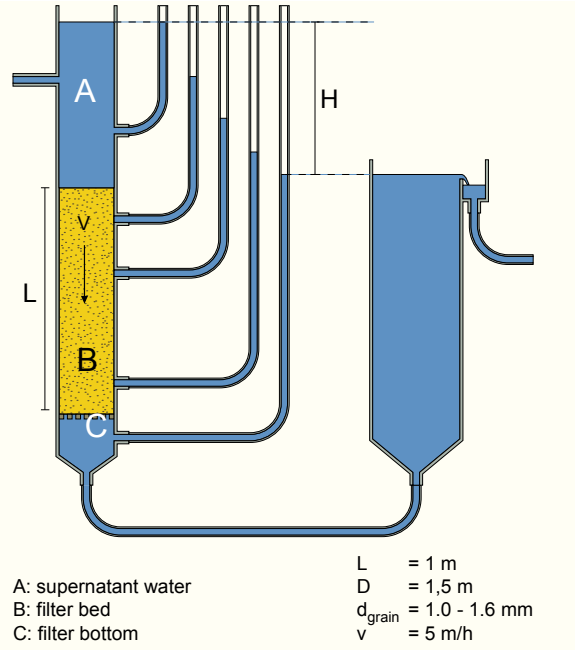



*Figure 4 - Filtration, experimental set-up*

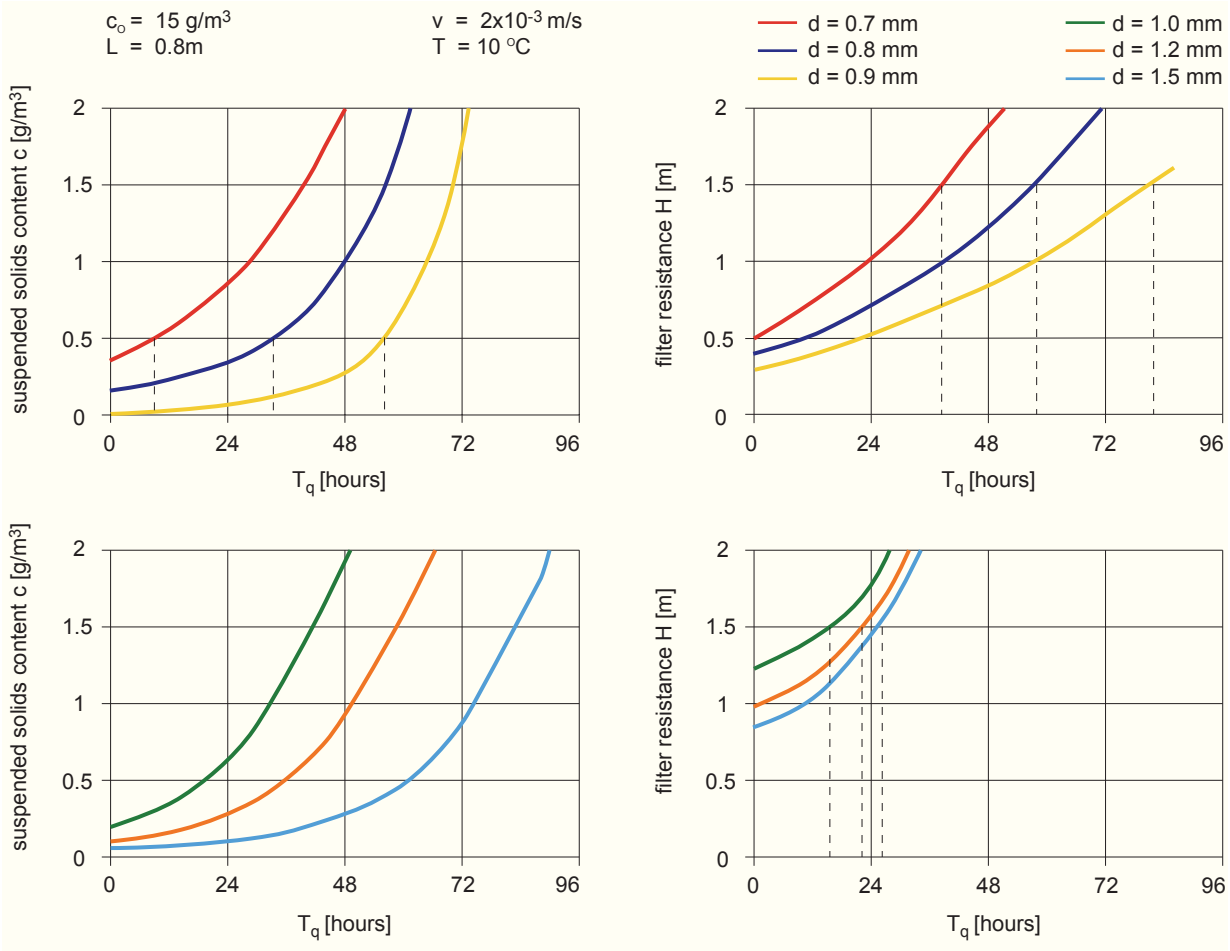



*Figure 5 - Results of different filter runs to obtain an optimally functioning filter*

filter (Figure 4).
In the test setup, the optimal combination of the following design parameters should be found:

- grain diameter of filter material
- filtration velocity
- height of the filter bed
- height of the supernatant water.

The optimal combination leads to a filter that is cheap, always satisfies the required effluent quality and has a reasonable filter run time. In addition, during the filter run, the suspended solids should be divided over the filter bed height, to avoid cake filtration.

The filter surface area has to be as small as possible to reduce investment costs.
Consequently, the filtration velocity has to be high. The higher the filtration velocity the sooner the effluent quality will deteriorate during the filter run. This can be compensated for by increasing the filter bed height or by choosing filter material with a smaller grain size.
A higher filter bed, however, means a higher filter construction and, thus, higher construction costs.
Filter material with a smaller grain size will clog faster, shorter filter runs will occur and operational costs will be increased.
In the graphs of Figure 5 the effluent quality and the filter resistance are represented for different filter mate-rials and for different filter bed heights.

In general it is assumed that the effluent quality has to remain under a determined effluent guideline.
The filter run time during which the effluent quality satisfies the guideline is called $T_q$.
Normally, filters are backwashed after run time $T_r$ , when a predetermined maximum resistance is reached.

To prevent the water quality from deterioratimg before the maximum resistance is reached, the filter design should fulfill the condition: $T_r<T_q$.
The aforementioned design parameters determine the values of both $T_q$ and $T_r$ (Figure 5).
In practice some restrictions are given to this optimization process.
Normally, safety margins are introduced to maintain the quality of drinking water above all suspicion and filter run times of 1 to 2 days are used.
In addition, a filter plant is always designed to take future developments into account. Consequently, most of the time a filter is operated below its capacity and far from the optimal situation.

## 3 Theory

### *3.1 Filtration*

Without making use of the filtration theory, a long series of filtration experiments would be necessary to come to an optimal solution for an installation, in practice.
This is problematic, because the raw water quality varies during the year and experiments would take at least a year to complete.
The filtration theory makes it possible to quantitatively predict the effects of changes in design parameters, based on the results of a reduced number of filtration experiments.

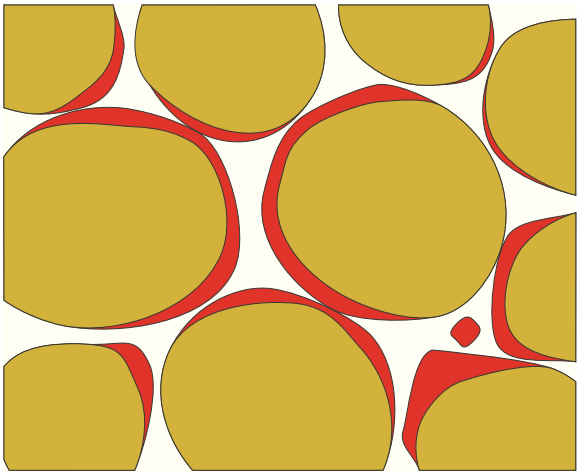


*Figure 6 - Reduction of pore volume as a result of accumulated solids*

**Effluent quality**

During the filtration process suspended and colloidal solids accumulate on the grains. Consequently, the concentration of suspended and colloidal solids decreases with the increasing filter bed depth.
In addition, the pore volume will be reduced in time due to the accumulation of suspended and colloidal solids, and the grain size of the filter material will be increased.
With a constant filtration rate (superficial velocity), the pore velocity will increase as filter clogging proceeds.

The equation that is formulated for filtration:

$$\frac{\partial c}{\partial t} = -u \cdot \frac{\partial c}{\partial y} - \lambda \cdot u \cdot c$$

together with the mass balance:

$$\frac{\partial \sigma}{\partial t} = -v \cdot \frac{\partial c}{\partial y}$$

in which:

| | | | |
|---|---|---|---|
| c | = | concentration of suspended and colloidal solids | [g/m$^3$] |
| y | = | depth of the filter bed | [m] |
| v | = | filtration rate | [m/s] |
| p | = | porosity | [%] |
| u | = | pore velocity (=v/p) | [m/s] |
| $\lambda$ | = | filtration coefficient | [m$^{-1}$] |
| $\sigma$ | = | accumulated solids | [g/m$^3$] |

In the stationary situation the following is valid:

$$\frac{\partial c}{\partial t} = 0$$

so the kinetics equation is transformed into:

$$\frac{\partial c}{\partial y} = -\lambda \cdot c$$

To solve the system of equations the value of the filtration coefficient $\lambda$ must be known.
However, $\lambda$ depends on different factors, such as the filtration velocity, viscosity, grain size, quality of

the raw water, and the clogging of the bed.
After start-up of the filtration process the filtration coefficient will initially increase because of better attachment characteristics on the preloaded material.
Due to pore clogging, the pore velocity increases and fewer solids will accumulate, expressed by a lower filtration coefficient $\lambda$.
When the solids are retained in the top layer of the filter bed, lower layers will take over until the filter is saturated and the filter breaks through.

The clean bed filtration coefficient $\lambda_0$ and the relationship between $\lambda$ and $\sigma$ have to be determined in practice (through column experiments).
Several researchers have found empirical relationships. Well-known relationships are those of Lerk and Maroudas.

Lerk

$$\lambda_0 = \frac{k_1}{\upsilon \cdot \nu \cdot d^3}$$

Maroudas:

$$\lambda = \lambda_0 \cdot \left(1 - k_2 \cdot \frac{\sigma}{\rho_d \cdot p_0}\right)$$

in which:

| | | |
|---|---|---|
| d | = grain size | [m] |
| $p_0$ | = initial porosity | [%] |
| $k_1$, $k_2$ | = constants | |
| $\nu$ | = kinematic viscosity | [$m^2/s$] |

The ratio between the accumulated solids $\sigma$ and the density is the reduction in pore volume ($\sigma_v$)

$$\frac{\sigma}{\rho_d} = \sigma_v$$

in which:

| | | |
|---|---|---|
| $\rho_d$ | = density of the flocs | [$kg/m^3$] |
| $\sigma_v$ | = volume concentration in pores | [$m^3/m^3$] |

The value of the constant $k_1$ is often assumed to be $9 \cdot 10^{-18}$ and the constant $k_2$ is the reciprocal value of the maximum pore filling n (0<n<1).

Notice that in the case of Madouras it is assumed that the filtration coefficient decreases linearly as clogging increases. Although this is a simplification, with this assumption the system of equations can be solved.
With the boundary conditions y = 0, $c = c_0$ and the initial condition t = 0, $\sigma_v = 0$ and:

$$\alpha = \frac{v \cdot c_0 \cdot \lambda_0}{n \cdot \rho_d \cdot p_0}$$

The solution becomes:
- general solution:

$$c = c_0 \cdot \frac{e^{\alpha \cdot t}}{e^{\lambda_0 \cdot t} + e^{\alpha \cdot t} - 1}$$

- effluent quality (y=L):

$$c_e = c_0 \cdot \frac{e^{\alpha \cdot t}}{e^{\lambda_0 \cdot t} + e^{\alpha \cdot t} - 1}$$

and:

$$\sigma_v = n \cdot p_0 \cdot \frac{e^{\alpha \cdot t} - 1}{e^{\lambda_0 \cdot y} + e^{\alpha \cdot t} - 1}$$

**Filter resistance**

During filtration pore clogging increases and, therefore, so does resistance in the filter bed.
When the filter reaches the maximum available head loss, the filter needs to be backwashed to avoid a decrease in the filtration velocity. The maximum available head loss is the difference between the supernatant water level and the head of the outflowing water, minus the clean bed resistance and head loss caused by filter bottoms, pipes and valves (Figure 7).

The clean bed resistance ($H_0$) can be derived from the equation of a flow through a pipe (pore) and can be described with the Carman-Kozeney equation:

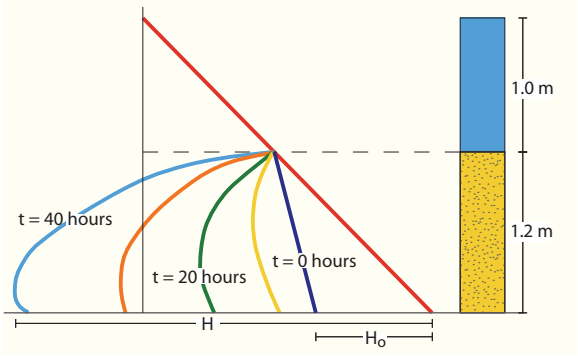



*Figure 7 - Progress of the filter bed resistance in time, the so-called Lindquist diagram*

$$I_0 = \frac{H_0}{L} = 180 \cdot \frac{v}{g} \cdot \frac{(1-p_0)^2}{p_0^3} \cdot \frac{v}{d_\lambda^2}$$

in which:
$I_0$ = initial resistance gradient [-]

This equation (the linear relationship between velocity and resistance) is only valid when:

$$Re = \frac{1}{p_0} \cdot \frac{v \cdot d_0}{\nu} \cdot < 5$$

When clogging occurs, the resistance formula changes into:

$$I = I_0 \cdot \left( \frac{p_0}{p_0 - \sigma_v} \right)^2$$

in which:
I = resistance gradient [-]

The solids accumulation in the pores $\sigma_v$ is known over the height and thus the resistance gradient can be calculated over the height of the filter bed.
By integrating the gradient the total resistance over the filter bed can be calculated.

As presented in Figure 7 the largest resistance is build up in the upper layers of the filter bed, where most of the solids accumulate.
In the lower layers the resistance gradient is almost equal to the clean bed gradient.

In time the resistance in the upper layers will increase.
A pressure drop in the filter bed below atmospheric (negative pressure) must be avoided. In such a case, dissolved gases will come out of solution and then released gas bubbles will disturb the filter bed.
Accumulated gas bubbles hinder downward water movement, increase filter resistance and end filter runs prematurely.
Negative pressure can be avoided by maintaining a high supernatant water level and shortening filter runs. This can be achieved by increasing the height of the outflow weir.

### 3.2 Backwashing

After a certain operation period the pores in a filter bed are filled with accumulated suspended solids. The porosity has decreased from $p_0$ to p, which results in a higher resistance and/or a poor effluent quality. Rapid filters are cleaned by backwashing with clean water (filtrate).

During backwashing the water flows in an upward

*Figure 8 - Filtration, backwashing with water and air*

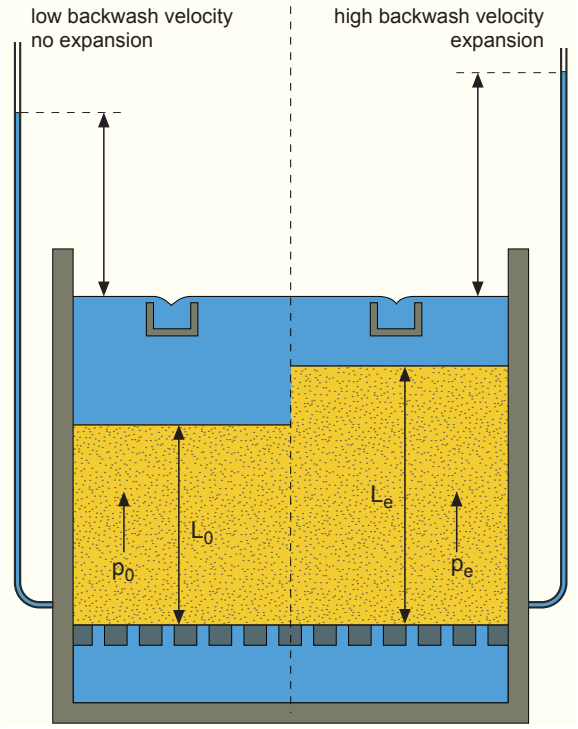



*Figure 9 - An expanded and non-expanded filter bed*

direction through the filter. The water scours the filter grains, erodes the accumulated solids from the filter material, expands the filter bed, and transports the solids towards the backwash troughs.

The shearing (scouring) force of the water is equal to the mass of grains under water:

$$\tau \cdot \pi \cdot d^2 = \frac{\pi}{6} \cdot d_3 \cdot (\rho_f - \rho_w) \Rightarrow \tau = \frac{d}{6} \cdot (\rho_f - \rho_w)$$

i n

which:

| | | | |
|---|---|---|---|
| $\tau$ | = | shearing force | [kg/m$^2$] |
| $\rho_f$ | = | mass density of the filter material | [kg/m$^3$] |
| $\rho_w$ | = | mass density of the water | [kg/m$^3$] |

The larger the diameter of the grains, the larger the shearing forces.
From practice, it is known that with grain diameters smaller than 0.8 mm backwashing is difficult. Therefore, a combination of water and air is used. By using air a more turbulent situation is created which facilitates the removal of the solids from the pores.

**Hydraulics of back washing**

Bed expansion is an important parameter for the design of a backwash facility:

$$E = \frac{L_e - L_0}{L_0}$$

in which:

| | | |
|---|---|---|
| E = bed expansion | | [-] |
| $L_0$ = initial height of the filter bed | | [m] |
| $L_e$ = height of the expanded filter bed | | [m] |

The applied bed expansion depends on the diameter of the filter material.
When the filter material has a diameter of 0.8 mm an expansion of 15 to 20% is used, while a diameter of 1.2 mm requires an expansion of 10%.
During backwashing, the amount of filter material remains constant (with a well-designed filter no losses of filter material occurs). When the initial porosity ($p_0$), the height of the filter bed during filtration, and the height during backwashing are known, the porosity during expansion can be calculated:

$$(1 - p_0) \cdot L_0 = (1 - p_e) \cdot L_e \Rightarrow p_e = \frac{p_0 + E}{1 + E}$$

in which:
$p_e$ = porosity of expanded bed

A backwash rate of 40 m/h through a filter bed with a porosity of 40%, a grain diameter of 1 mm and a temperature of 10 °C gives a Reynolds number of 14.1. Thus, the water flow during backwashing is no longer laminar, but situated in the transition zone between laminar and turbulent, and the Karman-Kozeney equation is not valid.
From experiments an empirical equation for the resistance during backwashing has been derived:

$$H = 130 \cdot \frac{\nu^{0,8}}{g} \cdot \frac{(1 - p_e)^{1,8}}{p_e^{\,3}} \cdot \frac{v^{1,2}}{d^{1,8}} \cdot L_e$$

This empirical equation is valid until the upward flow rate becomes so high that the bed fluidizes, the grains do not support each other and float.
Fluidization occurs when the resistance is equal to the mass of the filter bed under water:

$$\rho_w \cdot g \cdot H_{max} = (1 - p) \cdot L \cdot (\rho_f - \rho_w) \cdot g$$

.

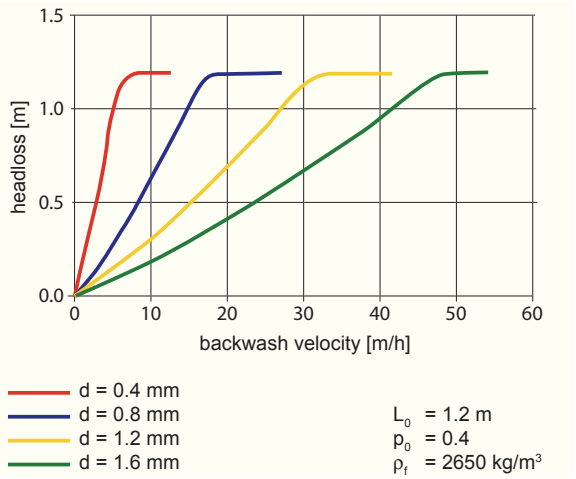



*Figure 10 - Resistance during backwashing a sand filter bed with a height of 1.2 m as a function of the backwash velocity and grain diameter*

$$H_{max} = (1-p)\cdot L\cdot\left(\frac{\rho_f-\rho_w}{\rho_w}\right)$$

When sand is used as filter material ($\rho_f$ = 2600 kg/m3 and p=0.4), the maximum resistance is almost equal to the bed height, since after substituting the values in the equation follows that H»L.

Figure 10 represents the resistance as a function of the backwash velocity. From the graphs it can be concluded that the maximum resistance is independent of the grain diameter.

The backwash velocity needed to achieve a certain expansion E and a resulting porosity pe can be calculated with a combination of equations given earlier:

$$(1-p)\cdot L\cdot\left(\frac{\rho_f-\rho_w}{\rho_w}\right)=130\cdot\frac{v^{0,8}}{g}\cdot\frac{(1-p_e)^{1,8}}{p_e^{\,3}}$$

From this equation it follows that:

*Table 1 - Backwash rates for obtaining a specific bed expansion at different temperatures for sand ($r_f$ = 2600 kg/m³, d = 1.0 mm, p = 0.38)*

| Temperature | Expansion | | | |
|---|---|---|---|---|
| | 0% | 10% | 20% | 30% |
| 0 | 16.2 | 24.5 | 33.5 | 42.8 |
| 10 | 20.2 | 30.2 | 41.4 | 52.9 |
| 20 | 23.8 | 36.0 | 49.0 | 63.0 |
| 30 | 27.7 | 41.8 | 56.9 | 73.1 |

$$v^{1,2}=\frac{g}{130\cdot v^{0,8}}\cdot\left(\frac{\rho_f-\rho_w}{\rho_w}\right)\cdot\frac{p_e^{\,3}}{(1-p_e)^{0,8}}\cdot d^{1,8}$$

In Table 1 the backwash rate for sand is given under variable bed expansions and water temperatures. To achieve a bed expansion of 20% a backwash rate of 30 m/h is necessary. This backwash rate is considerably higher than the filtration rate that varies from 1 to 20 m/h (for rapid filtration).

## 4 Practice

### *4.1 Backwashing*

When a rapid filter is backwashed, the water supply (valve A in Figure 11) is stopped. The supernatant water still filtrates through the bed.
After the filtrate drainage is blocked (valve B), the backwash process is started. That is when valves C and D are opened. The filter is backwashed for a certain period of time with water and air. When the bed is sufficiently clean, the supply of water needed for backwashing is stopped and the wash water drain is closed by valve D.
By opening valve E the supernatant water filtrates through the bed. Then valve A is opened. Since the water that leaves the filter during the ripening

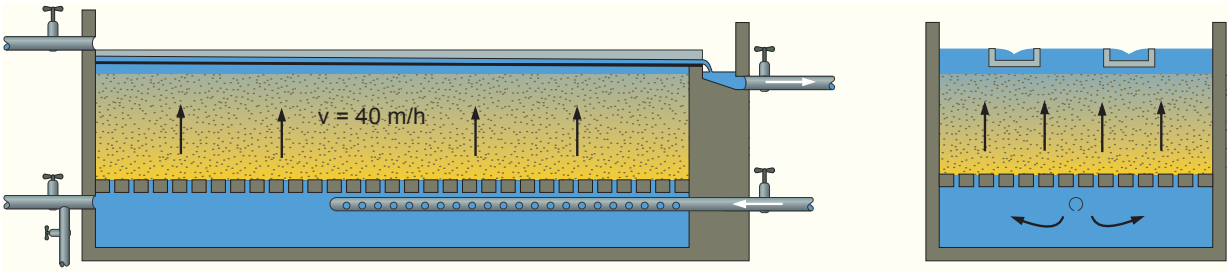



*Figure 11 - Schematic representation of the backwash process*

period is generally of poor quality, this water is drained into a waste receptacle.
After the ripening period valve E is closed and B is opened.
The total time that a filter is not in operation due to the backwash procedure varies from 30 to 60 minutes. The backwashing process itself will last approximately 20 minutes.
When the filter run and the backwash time are known, the net production through the filter can be calculated.

Over a short period of time a high wash water flow is needed. There are two possibilities to supply these flows:
- backwash pumps
- elevated water reservoir.

When using backwash pumps, water is obtained directly from the filtrate reservoirs.
During a short period, high energy consumption takes place. This is expensive. The pipes that transport the wash water from the filtrate reservoirs to the filters are the largest pipes in a water treatment plant.
Assuming that the maximum permitted velocity in a pipe is 1 m/s, it can be calculated that the diameter of the backwash supply pipe to a filter with a surface area of 80 $m^2$ and a backwash rate of 50 m/h is almost 1.2 meters.
The diameter of the supply pipe for raw water is much smaller. When the filtration rate is 5 m/h, this diameter is 0.35 meter.

A filter with a surface area of 80 $m^2$, a filter run time of 72 hours and a filtration rate of 5 m/h is backwashed for 20 minutes with a backwash rate of 50 m/h. In addition, the filter is not operating during 20 minutes due to drainage of supernatant water and also due to filter to waste.
The filtrate production is:
(72-40/60)*5*80=28.533 $m^3$.
The quantity of filtrate used for backwashing is 0.333*50*80=1.333 $m^3$, that is a water loss of 5%.

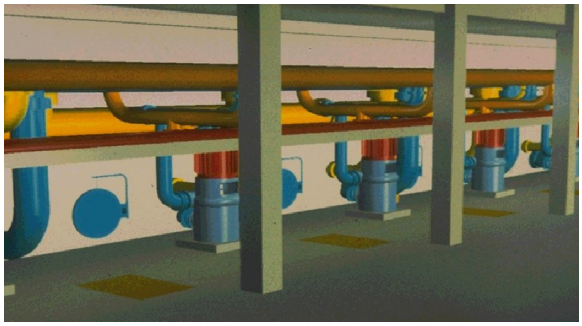


*Figure 12 - Supply pipes for backwash water are the largest pipes in a filter installation*

When an elevated water reservoir is used, backwash pumps with a lower capacity (10 to 20% of the backwash capacity) can be applied. These backwash pumps continuously fill the reservoir.
A disadvantage, however, is that a separate elevated water reservoir needs to be built, thereby increasing the investment costs.

The required pressure for backwash water is around 2-5 mWc. In addition to the static elevation height, most of the pressure losses are due to the resistance that is a result of the flow of water through the filter bottom, the filter bed, and the piping system.

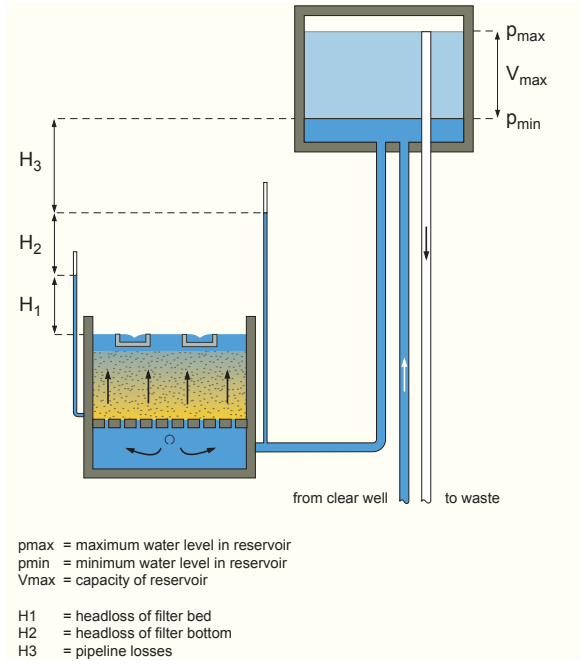



*Figure 13 - Elevated water reservoir as backwash system*

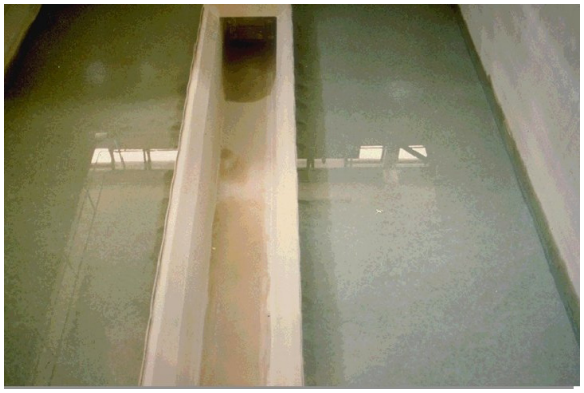


*Figure 14 - A central backwash trough*

After passing the filter bed, the wash water is drained through a system of troughs.
The troughs are designed to limit the (horizontal) distance the water must travel after leaving the filter bed.
The moment the water leaves the filter bed, the wash water velocity decreases by a factor of 2.5 and settling in the supernatant water can occur.

The right configuration of backwash water troughs is found by optimization. The more troughs, the higher the investment costs, but the lower the wash water loss is.
In practice, it turns out that troughs at the sides and troughs at the front side are satisfactory and cheap.

In large filters, a “water sweep” is applied to reduce the volume of backwash water. Raw water is supplied from the side onto the filter bed. Thus, the water flows from the sides to a central trough, resulting in a stable flow without short circuits and eddies.
In the past wash water was drained to waste water ponds, the solids would settle and the supernatant water was drained to the surface water or sewerage system.
Because of stricter regulations concerning discharge to surface waters, soil protection and groundwater protection measures, backwash water ponds are not used anymore. These days, backwash water is transported to backwash water treatment installations.
In such installations, a coagulant is added to the backwash water, resulting in floc formation incorporating the solids. The flocs are removed afterwards by titled plate separators and rapid sand filters. After UV-disinfection the treated water can be recycled into the main treatment process.
An alternative backwash water treatment plant consists of micro-/ultrafiltration.

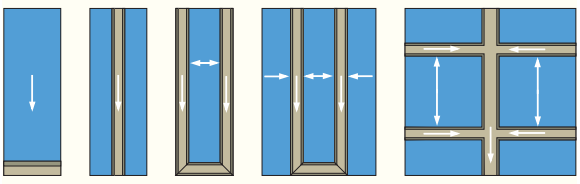


*Figure 15 - A central backwash water trough*

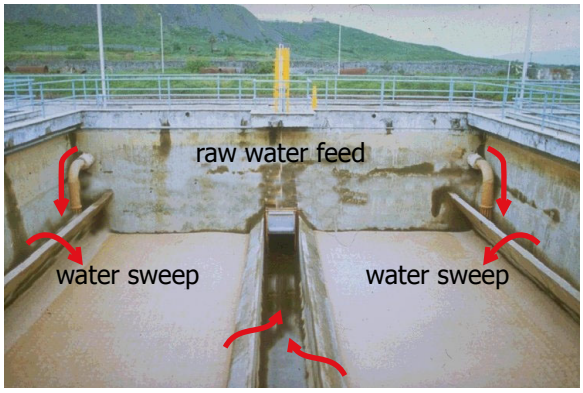



*Figure 16 - Water sweep*

### 4.2 Filter bottom

Water that passes the filter bed is drained through nozzles to the filtrate reservoir.
Nozzles are synthetic tubes that are incorporated into the construction of the filter bottom.
To avoid loss of filter sand, perforated heads are placed onto the tubes (Figure 17 and 18).
Frequently, a number of support layers of coarse filter material are placed between the filter bottom

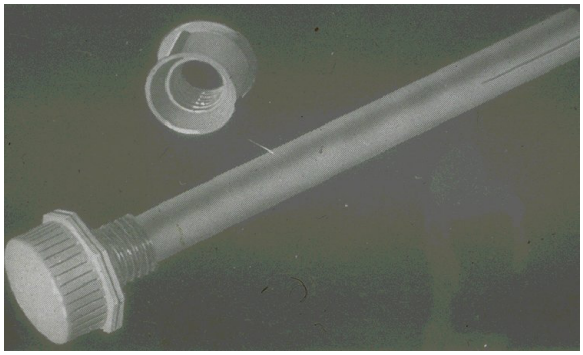


*Figure 17 - Nozzle*

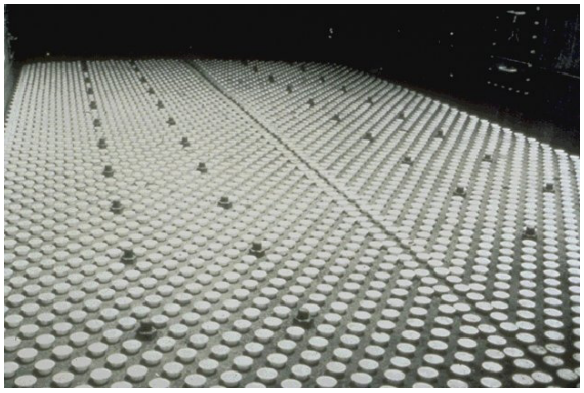


*Figure 18 - Nozzle bottom*

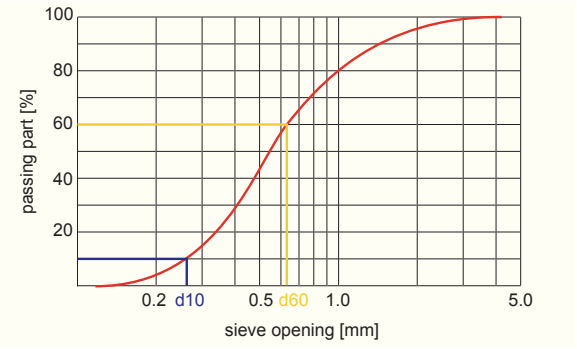



*Figure 19 - Sieve curve of filter sand*

and the filter bed to enable larger slot sizes in the nozzle, for avoiding clogging of the nozzles.

In addition to draining the filtered water, nozzles also have a function in the supply of backwash water and air.
The most important aspect is a uniform distribution of water and air over the filter bed, which can be achieved by introducing a considerable filter bottom resistance (0.5-2 m).

### *4.3 Filter material*

Not all granular material can be used as filter material.
The material should have certain characteristics, such as:

- resistant to abrasion (wear)
- free of impurities
- uniform grain-size distribution.

Typically, river sand is applied as filter material. River sand has a great variety of grain sizes and must therefore be sieved before application. The uniformity of the filter material can be expressed in the uniformity coefficient, defined as:

$$U = \frac{d_{60}}{d_{10}}$$

in which:

U = uniformity coefficient [-]
$d_{10}$ = size of sieves that let pass 10% of the sand mixture [mm]
$d_{60}$ = size of sieves that let pass 60% of the sand mixture [mm]

If the uniformity coefficient equals 1, the material is uniform. A higher coefficient indicates a larger variety in the grain sizes (Figure 19).
For rapid filtration the value of the uniformity coefficient should be between 1.3 and 1.5 to avoid stratification of the filter bed during backwashing. A lower value of the coefficient is possible, but this results in higher sieving costs and provides little additional advantage.

Other filter materials are given in Table 2.
Filter material with a low density is used when the diameter of the filter material should be large and the backwash rate is limited.
The heavier filter materials are used during upflow filtration to avoid premature expansion of the filter bed.

### *4.4 Filter troubles*

In spite of rapid filtration being a simple process, many problems can occur.
The choice of the filter material and the design of the filter bottom are crucial.

*Table 2 - Density of different filter materials*

| Filter material | Specific density |
|---|---|
| Plastic grains | 1050-1300 |
| Pumice | 1200 |
| Anthracite | 1400-1600 |
| Sand | 2600 |
| Garnet | 3500-4300 |
| Magnetite | 4900-5200 |

When the filter material is badly sieved and thus not uniform, stratification will occur during the backwash process. Hence, the lighter, smaller grains will collect at the top of the filter bed, and the heavier, larger grains will settle on the bottom of it.
During filtration all suspended solids will accumulate in the fine upper layer. This phenomenon is called surface or cake filtration. The cake is difficult to remove during regular backwash procedures. Cracks will be formed in the cake, and preferential flows will occur. In addition, mud balls will form and settle on the filter bottom, clogging the nozzles.

When a stratified bed is backwashed with low velocities, only the upper layer will be expanded (with the small grain sizes). The lower layers will not or will hardly be expanded and accumulated solids will not be removed. A larger backwash rate will result in a washing out of the upper layers.

A non-uniform flow during backwashing (by a poor design or clogging) can lead to preferential flows in and disturbance of the filter bed.
This can result in total mixing of the filter material, and support layers that must be situated at the bottom can be found at the surface. This phenomenon is called sand boil.

The filtrate and the wash water must be able to pass through the filter bottom, but filter material must be retained.
Because the grain size of the filter material is about 1 mm, a small crack in the bottom is large enough for the grains to pass through it and for the filter to become a huge sandglass (Figure 20)
These cracks can be caused by damage in the nozzle or inaccurate sealing of the bottom plates.

*Figure 20 - Result of a poorly designed filter bottom*

## 5 Alternative applications of filtration

### *5.1 Multiple layer filtration*

Multiple layer filtration consists of a filter bed with various layers of different grain sizes. The water passes the coarser grains first, resulting is a tapered filtering.
In upflow filters, the coarse grains are at the bottom (Figure 21).

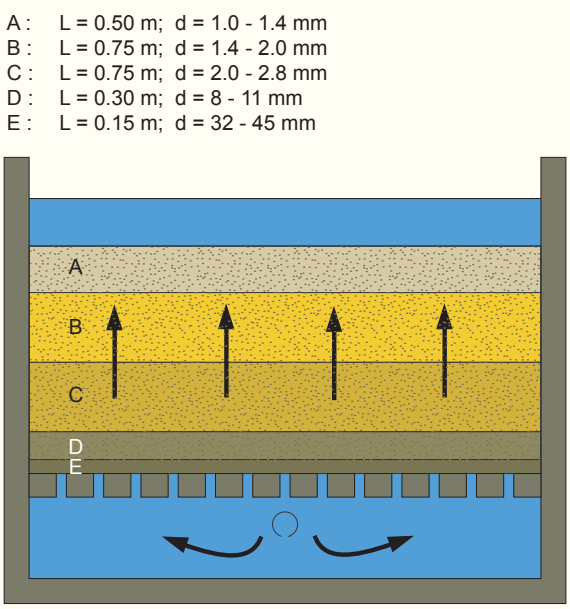



*Figure 21 - Multiple layer filtration (upflow filter)*

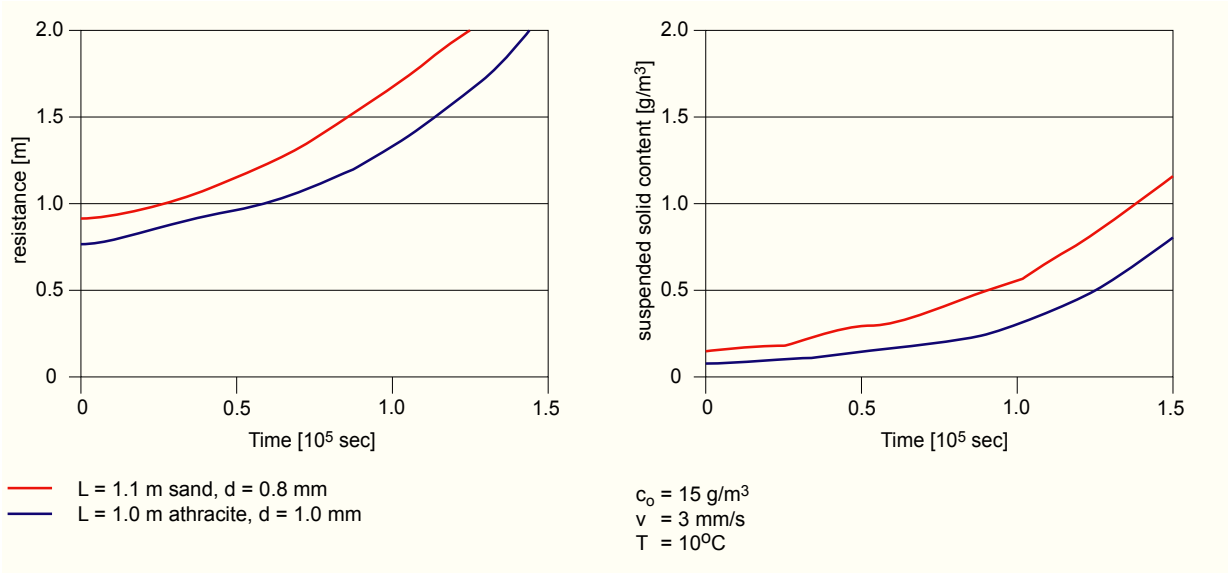



*Figure 22 - Difference between single and multiple layer filtration for equal bed heights*

In downflow filters the grain size decreases in downward direction.
The size and the density are chosen so that the settling velocity of the material in the bed increases in a downward direction and mixing between the two layers during backwashing does not occur.
Usually sand is used as filter material in combination with either:

- a filter layer with a large grain size and a low density on top of the sand (anthracite)
- a filter layer with a small grain size and a high density below the sand layer (garnet).

Multiple layer filtration has an advantage that the larger solids are retained in the top layer of the bed and the smaller ones in the lower parts of the bed.
Consequently, the increase in filter resistance is spread over the entire height of the bed, resulting in an extended filter run time.

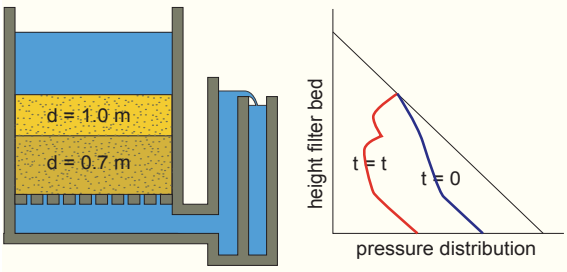



*Figure 23 - Resistance build-up for a multiple layer filter*

In Figure 22 the progress of the water quality and the resistance are represented for a single layer and a double layer filter bed.
Using the same filtration velocity the resistance of a double layer bed is lower than of a single layer bed and the effluent water quality is better.

## 5.2 Pressure filtration

Pressure filters are based on the same principles as gravity rapid filters. The only difference is that the filter bed with the supporting filter bottom and the supernatant raw water are encased in a water-tight steel cylinder. This gives a closed system in which the water to be treated can be forced through the filter bed under pressure.
On one hand, this high pressure allows a large filter resistance without the danger of negative heads; while on the other hand, filtered water pumps are no longer required and the filter can be placed at any random level. Hence, the hydraulic head does not have to be considered.

In addition, the application of a large filter resistance permits the use of high filtration rates through filter beds with long filter run times ($T_r$).
The filtration rates normally vary from 7 to 20 m/h, while values of 55 m/h are no exception. The surface area of a pressure filter can thus be small.
The contact time between the water to be treated

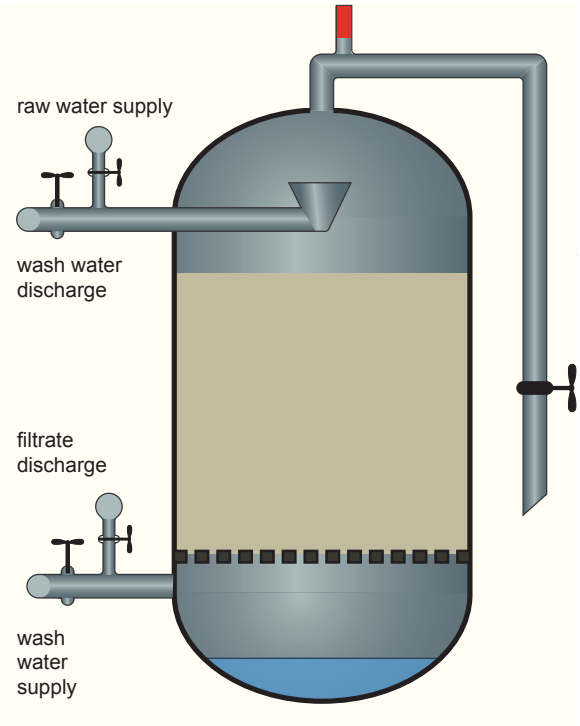



*Figure 24 - Principle of pressure filtration*

The height above the filter is determined by the distance between the drainage troughs and the filter bed. This distance varies between 0.4 to 0.6 meters.

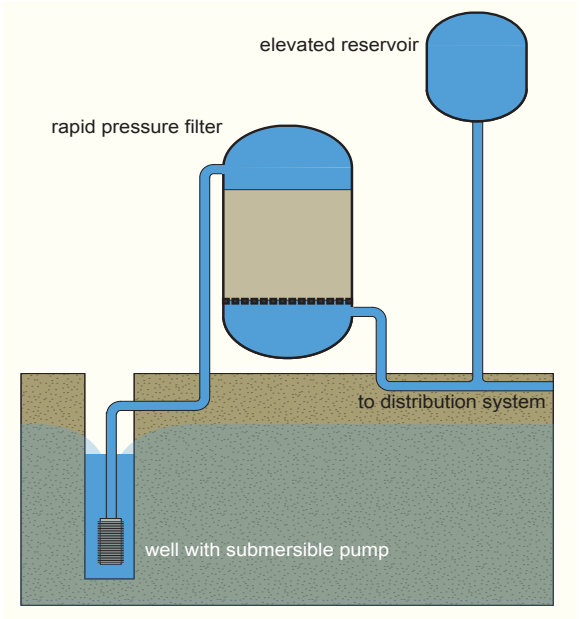



*Figure 25 - The filtered water flows without a pumping phase towards the next treatment step*

*Figure 26 - Cylindrical steel pressure filters*

### 5.3 Upflow filtration

The longest filter runs and the best water quality are obtained when water passes first a coarse fraction and afterwards a finer fraction of the filter material.
In upflow filtration the coarse material is situated at the bottom and the fine material at the top.
During backwashing and filtration the filter bed is conserved as a result of (natural) stratification.
The elevation height of the water is equal to the hydrostatic water pressure plus the resistance due to flow and clogging. This resistance is the largest in the bottom of the bed (y=L).

Fluidization of sand with a density of 2600 kg/m$^3$ and a porosity of 40% will occur when the resistance is approximately equal to the height of the filter bed.
If fluidization occurs too quickly, a higher filter bed or a filter material with a higher density can be applied.

In addition to the aforementioned advantage of good effluent quality, up flow filtration has several disadvantages:

- wash water and filtrate are drained with the same trough. This increases the risk of contamination of the filtrate
- fluidization of the top layer of the filter bed can occur, resulting in wash out of filter material, diminishing the filter bed height and lowering the removal efficiency
- raw water is uniformly distributed by nozzles in the bottom of the filter. The nozzles can clog by impurities in the raw water which results in

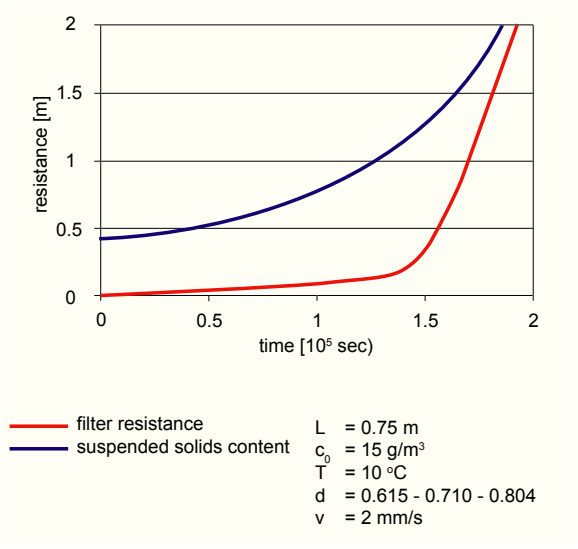



*Figure 27 - Changes in quality and resistance of an upflow filter*

extra resistance and a non-uniform distribution of the water over the filter bed.

### 5.4 Limestone filtration

Limestone filters are filled with grains of calcium carbonate or half-burned dolomite. When aggressive water (with high levels of carbonic acid) passes these filters, the concentration of carbonic acid will decrease and the levels of hydrogen carbonate and pH will increase.

Water that is not in (calcium-carbonic acid) equilibrium dissolves limestone grains (calcium carbonate) according to the reaction:

$$CaCO_3 + CO_2 + H_2O \longleftrightarrow Ca^{2+} + 2{\cdot}HCO_3^-$$

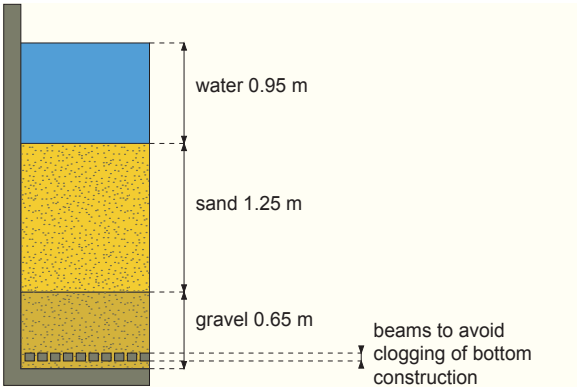



*Figure 28 - Principle of uplow filtration*

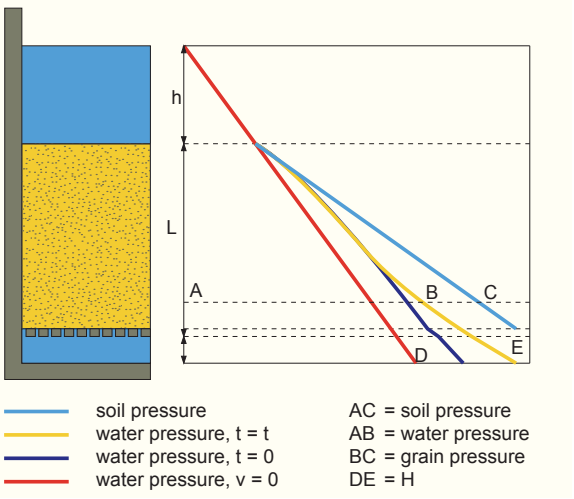



*Figure 29 - Pressure distribution in an upflow filter*

Because the limestone grains are dissolved, they need to be replenished regularly.
Normally, replenishing is executed when 10% of the limestone is used. If limestone filtration is used in groundwater treatment after aeration, ferric and manganese removal and nitrification can occur in the filter.

### 5.5 Continuous filtration

In a continuous filter sand is re-circulated and purified by a pump centrally placed in the filter. Hence, the backwash process can be continuous.
In a continuous filter the water flows in an upward direction, transport of sand occurs in a downward direction (Figure 30). The inlet of raw water (with impurities) is at the bottom of the filter. At the top of the filter the water flows over a weir and is transported to other treatment processes.

The sand retains the impurities. By means of a sand pump (mammoth pump), the lowest layers of sand are removed from the filter and brought to a sand washer that is situated above the filter. The sand washer removes the impurities from the sand and the clean sand is supplied on top of the continuous filter.

Due to the continuous removal of impurities, the quality of the filtrate, the bed resistance and the pressure distribution in the filter are constant and time independent. The filtration rates of a continuous filter vary from 14 to 18 m/h.

The advantages of a continuous filter compared to a rapid sand filter are:
- continuous filtration process
- continuous wash water flow
- less accumulation of sludge.

The disadvantages of a continuous filter compared to a rapid sand filter are:
- large wash water flows
- sand wash installation is in direct contact with filtrate, resulting in contamination risks.

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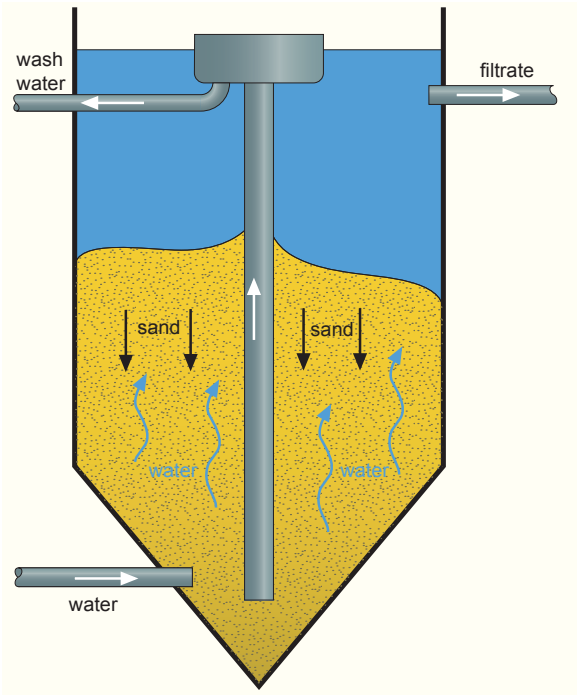


*Figure 30 - Schematic representation of a continuous filter*

## 5.6 Dry filtration

Dry filtration is used when the water contains a high ammonia concentration. Thus, dry filtration is only applied in river bank and groundwater treatment.

The oxidation of ammonia into nitrate requires large amounts of oxygen: 3.55 mg/l $O_2$ per mg/l $NH_4^+$. The oxygen concentration of water is approximately 10 mg/l. Hence, in water with ammonia concentrations larger than 2.5 mg/l, nitrification will be incomplete.

Dry filtration has no supernatant water level. The water to be treated flows in a downward direction through a bed of granular material, accompanied by a downward or upward flow of air of about the same magnitude. A continuous gas transfer between air and water will take place. The oxygen consumed during the treatment can be replenished directly by the accompanying air. The formed carbon dioxide is removed from the water. The pores are only partially filled with water and, thus, the velocity of the water through the pores is greater than in rapid sand (wet) filtration. The flow conditions through the pores are turbulent thereby promoting the hydrodynamic transport of impurities from the flowing interstitial water to the filter grain surfaces where they attach.

The filtered water collects below the filter bottom and flows via gravity to the next treatment process. From the filtrate chamber, air is continuously pumped by a ventilator maintaining a (forced) simultaneous flow of air through the filter bed (Figure 31). When, in addition to oxygen transfer, the dry filter is also used for gas stripping, a counter-current flow of water and air can be used in the filter (Figure 32).

The water to be treated is distributed over the full area of the dry filter bed as evenly as possible with the help of spray nozzles.

Spraying has two objectives:
- gas transfer (addition of oxygen and removal of methane and carbonic acid)
- uniform distribution of the water over the filter.

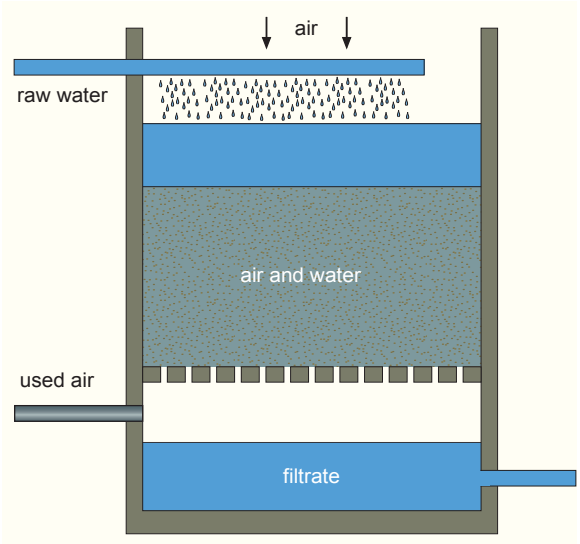


*Figure 31 - Schematic representation of a co-current filter*

A dry filter does not only remove ammonia, but also iron and manganese. In the top layer of the filter bed (depth of 0.5 to 1.5 m) iron removal takes place. After completion of this process, manganese and ammonia removal occurs more or less simultaneously. Dry filters are often followed by rapid filters. The reason is that in a dry filter bacteria are formed. The rapid sand (wet filter) forms a barrier against the breakthrough of these bacteria.

### *5.7 Slow sand filtration*

When the most important objective of a filter is to remove bacteria and viruses and the filter is an alternative for chemical disinfection, slow sand filters are suitable.
The filter material has a small grain size (e.g., 0.2 to 0.6 mm) and the filtration rate is below 1 m/h.
For treatment of the same water flow a larger filtration surface area is needed than used for rapid filters. This is illustrated in the aerial picture of the treatment plant at Leiduin (Figure 33).

Filtration occurs mainly in the top layer of a slow sand filtration, where a biologically active “Schmutzdecke” is formed.
To clean the filter, backwashing cannot be applied. Instead, the upper sand layer (usually 1 cm) is scraped.

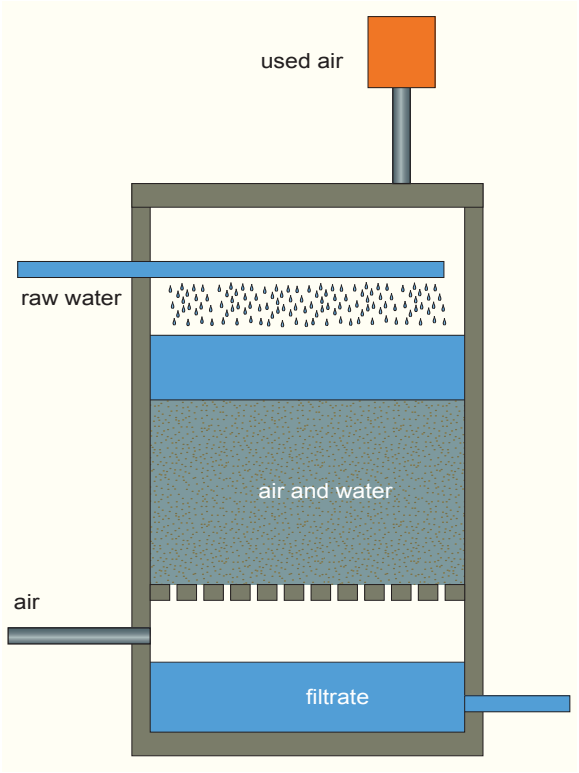


*Figure 32 - Schematic representation of a counter-current dry filter*

Because slow filters are normally placed after rapid filters, slow filters are barely loaded with impurities and the filter run time can have an order of magnitude of several years.

*Figure 33 - Difference in surface area between rapid filtration and slow sand filtration*

ARAB REPUBLIC OF EGYPT
German Technical Cooperation

# تقنيات معالجة مياه الشرب

# Drinking Water Treatment Technology

## الماده المقدمه

## Presentation

**CH / Mohamed Shaltout**
**Eng / Mohamed Ghonam**

November 2008

**Deutsche Gesellschaft für Technische Zusammenarbeit - GTZ**
**Water and Wastewater Management Programme**
**GTZ Project No. 06.2006.3**

# المحتويات

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٣. عدد المتدربين
٤. منهجية التدريب
٥. مساعدات التدريب
٦. قائمة الدورات التدريبية
٧. مكان التدريب و طريقة الجلوس بجلسات التدريب

**ثانيا: خطة التدريس بالدورة التدريبية تقنيات معالجة مياه الشرب**

١. أهداف الدورة
٢. موضوعات الدورة
٣. مدة الدورة
٤. البرنامج الزمنى للدورة

## أولا: نظرة عامة على البرنامج التدريبي تقنيات معالجة مياه الشرب

### ١. الهدف العام للدورة التدريبية

يقوم علي إنتاج مياه الشرب فريق واحد هو فريق العاملين بمحطات المعالجة ( مهندسين وكيميائيين ) ولكن بين أفراد الفريق الواحد تختلف الرؤية حول التشغيل و الأولويات الواجب الاهتمام بها فيرى المهندسون أن كمية الإنتاج هي الهدف والمراد من المحطة لسد احتياجات المواطنين وإنهاء شكوى الأهالي و للتخلص من الضغوط السياسية و تأتي التكلفة الاقتصادية للمحطة لتكون معيار النجاح .

بينما يرى الكيميائيون أن جودة المياه المنتجة ومطابقتها للمواصفات هي الهدف والغاية و أن ارتفاع جودة المياه المنتجة لتقارب أعلي المواصفات هو مؤشر النجاح .

وتهدف الدورة إلى تعريف العاملين بمجال مياه الشرب معني معالجة المياه لتقريب وجهات النظر ونوضح للمهندسين والكيميائيين معني المعالجة والعمليات المرتبطة بها و ما يؤثر علي الإنتاج من حيث الكم والكيف وما هو مؤشر نجاح عمليات الإنتاج و المعالجة.

### ٢. المجموعة المستهدفة

٣. العاملين بالمعامل المركزية لمياه الشرب فى شركة مياه قنا وكذلك العاملين بمعامل المحطات و الترشيح السريع

### ٤. عدد المتدربين

يبلغ عدد المتدربين المقدر لحضور كل الدورة ما بين ٣٥ – ٤٠ متدرب من المعمل المركزى ومعامل المحطات بشركه قنا لمياه الشرب والصرف الصحى ..

### ٥. منهجية التدريب

تعتمد منهجية التدريب بالدورة على عدة اسس يكون الهدف الرئيسى منها توصيل المعلومة بسهولة ويسر للمتدرب وكذلك ضمان المشاركة الفعالة من المتدربين أثناء جلسات التدريب والتأكد من الفهم الكامل لمحتويات وموضوعات الدورة والتدريب العملى والشخصى على الموضوعات التى ستتناولها الدورة.

هذا ويمكن تلخيص المنهجية المتبعة فيما يلى:

- **المحاضرات:** التى يلقيها المدرب ذا الخبرة بهدف توصيل أحدث المعلومات على صورة نظرية وعملية والتأكد من التطبيق العملى بطريقة صحيحة وعلى أساس من

الفهم مما يمكنه من تلاشى الأخطاء التى من الممكن أن تلعب دورا فى صحة النتائج التى يتحصل عليها والتى تهتم بنوعية مياه الشرب.

- **الشرائح Power point:** التى تعرض أثناء الشرح لإبراز النقاط الرئيسية لكل موضوع فى تسلسل منطقى ولضمان وتثبيت المعلومة لدى المتدرب
- **المناقشات المفتوحة:** ويديرها المدرب أو المحاضر وتتيح هذة المناقشات الفرصة لتبادل الأراء وتوجبه الأسئلة و الحصول على معلومات جديدة كما إنه يتم من خلالها نقل المعارف والخبرة العملية والنظرية من المدرب إلى المتدربين واصلاح لمفاهيم غيرالصحيحة أو غير الحديثة لدى المتدربين.
- **دراسة الحالات الواقعية:** وهى تفيد فى عرض المشاكل العملية التى يواجهها المتدربون أو التى سوف يواجهونها فى عملهم و أساليب التغلب عليها بالاسلوب العلمى الصحيح.

- **التدريب العملى:** والذى سيتاح بصورة فردية لكل متدرب باستخدام الطرق القياسية الحديثة لضمان الفهم التام والتطبيق الصحيح من المتدرب للمعلومات والطرق العملية التى تم تدريسها.
- **المراجع العلمية و الكودات و المواصفات:** يتم إعطاء المتدرب المراجع العلمية التى أعتمد عليها والتى يمكن الرجوع إليها لزيادة التعمق فى المجال وكذلك الإشارة ومناقشة الكود الخاص بتشغيل محطات معالجة مياه الشرب والمواصفات الحديثة الحاكمة والمعمول بها فى مصر وعلى المستوى الدولى فى مجال مياه الشرب.

**فى نهاية الدورة يتم تقييم الحاضرين من خلال اختبار تحريرى فى مواد الدورة.**

## ٦. <u>مساعدات التدريب</u>

- جهاز عرض الشرائح (Power Point Projector)
- سبورة بيضاء أو سبورة ورقية
- شاشات عرض.

## ٧. مكان التدريب و طريقة الجلوس بجلسات التدريب

يجلس المتدربون وفى مواجهتهم المحاضر فى المنتصف وعلى يمينه جهاز الكمبيوتر لعرض الشرائح Power Point وشاشة العرض وعلى يساره السبورة البيضاء أو السبورة الورقية ويكون وضع كل من شاشة العرض والسبورة بحيث يسمح بسهولة الرؤية لجميع المتدربين.

وتقدر المساحة المطلوبة لقاعة التدريب بما لا يقل عن ٥ × ٧ مترا لتستوعب المتدربين والمدرب لتسمح بسهولة حركة المدرب وإمكانية وصولة لأماكن جلوس المتدربين. ويلزم أن تتوفر بالقاعة الإضاءة اللازمة والتهوية الكافية والأجهزة الصوتية المناسبة.

## ثانيا: خطة التدريس بالدورة التدريبية

**أ. دورة الطرق القياسية فى تجميع مياه الشرب**

**محاضر: ك/ محمد شلتوت**

**تدريبات عملية: مساعد من هيئة العاملين بالمحطة التى تتم فيه الدورة.**

**ب. دورة تقنيات معالجة مياه الشرب**

**محاضر: م/محمد غنيم**

**تدريبات عملية: مساعد من هيئة العاملين بالمحطة التى تتم فيه الدورة.**

**١ - أهداف الدورة**

بنهاية الدورة فإن المشاركين سوف:

- يكونون قادرين علي تقييم مراحل المعالجة المختلفة
- لديهم المنظور الهندسي لعمليات إنتاج المياه
- أحاطوا بالتقنيات المختلفة لمعالجة مياه الشرب معرفة الطرق التقليدية لمعالجة مياه الشرب( مقدمة)
- مصادر المياه الجوفية
- التكون الطبيعى للمياه الجوفية
- مصادر تلوث المياه الجوفية
- الطريقة الصحيحة للحصول علي المياه الجوفية
- اختيار موقع البئر
- الاحتياطات الواجب اتخاذها أثناء دق الآبار أو اقامتها
- حماية موارد المياه الجوفية
- مزايا وعيوب المياه الجوفية
- المعالجة التقليدية لمياه الشرب
- عملية الترويب و التنديف
- الترسيب
- اختبارات التحكم فى التشغيل
- المواصفات القاسيه المصرية ٤- ١٩٨٢ لكبريتات الالمونيوم ( الشبه الصلبة) المستعملة فى عمليات معالجة مياه الشرب
- تطهير المياه

**٢ - موضوعات الدورة**

- المقدمة وشرح الهدف من الدورة.

- المياه المصادر والخصائص.
- أنظمة معالجة مياه الشرب.
- عمليات معالجة .مياه الشرب
- الترويق clarification
- الترشيح Filtration
- زيارة محطة إنتاج مياه الشرب وتقييم مختلف مراحل المعالجة. مقدمة فى معالجة مياه الشرب
- المياه الجوفية
- التكوين الطبيعي للمياه الجوفية
- تلوث المياه الجوفية
- الطريقة الصحيحة للحصول علي المياه الجوفية
- المصادر االتى تؤدى الى تلوث الآبار
- حماية موارد المياه الجوفية
- مزايا وعيوب المياه الجوفيه
- المعالجة التقليدية لمياه الشرب
- الترسيب
- عملية الترويب و التنديف
- اختبارات التحكم فى التشغيل
- حساب جرعة الشبه
- المواصفات القاسيه المصرية ٤- ١٩٨٢ لكبريتات الالمونيوم ( الشبه الصلبة) المستعملة فى عمليات معالجة مياه الشرب
- تطهير المياه
- نظرية عمل لالكلور
- العوامل المؤثرة فى عملية التطهير

**٣ - مدة الدورة**

تستغرق الدورة مدة سته أيام متواصلة و يبدأ العمل يوميا من الساعة التاسعة صباحا حتى الساعة الرابعة بعد الظهر، أى مدة سبعة ساعات يوميا يتخللها نصف ساعة لتناول المشروبات والغداء.

## أ.٤ البرنامج الزمنى للدورة

| اليوم | الجلسة | التوقيت | الموضوع | المحتوى |
|---|---|---|---|---|
| اليوم الأول | جلسة الإفتتاح | ٩ – ٩.٣٠ | إستقبال و تسجيل المشاركين فى الدورة | |
| | الجلسة الأولى | ٩.٣٠-١١.٣٠ | مقدمة فى معالجة مياه الشرب | • الطرق التقليدية فى معالجة مياه الشرب<br>• خطوات معالجة مياه الشرب |
| | | ١١.٣٠-١٢.٠٠ | استراحة | |
| | الجلسة الثانية | ١٢.٠٠-٢.٣٠ | المياه الجوفية | • التكوين الطبيعى للمياه الجوفية<br>• تلوث المياه الجوفية<br>• الطريقة الصحيحة للحصول علي المياه الجوفية |
| | الجلسة الثالثة | ٢.٣٠-٤.٠٠ | المياه الجوفية | • المصادر االتى تؤدى الى تلوث الأبار<br>• حماية موارد المياه الجوفية<br>• مزايا وعيوب المياه الجوفيه |
| اليوم الثانى | الجلسة الرابعة | ٩.٠٠-١١.٣٠ | المعالجة التقليدية لمياه الشرب | • سجلات تسلسل الحيازة وجمع العينات وطلب التحاليل ونتائج التحاليل والتقارير |
| | | ١١.٣٠-١٢.٠٠ | استراحة | |
| | الجلسة الخامسة | ١٢.٠٠-٢.٣٠ | الترسيب | • الترسيب الطبيعى<br>• الترسيب باستخدام المروبات<br>• استخدام الشبة كمروب كيميائى |

| اليوم | الجلسة | التوقيت | الموضوع | المحتوى |
|---|---|---|---|---|
| | الجلسة السادسة | ٢.٣٠-٤.٠٠ | الترويب و التنديف | • نظرية تكوين الندف<br>• استخدم تجربة ال Jar test لتحديد الجرعة المثلى من المروبات<br>• الطرق الكيميائية لتحليل الشبة( صلب و سائل) |
| اليوم الثالث | الجلسة السابعة | ٩.٠٠-١١.٣٠ | مقدمة فى تطهير المياه | • المعالجة الحرارية<br>• المعالجة الاشعاعية<br>• المعالجة الكيماوية |
| | | ١١.٣٠-١٢.٠٠ | استراحة | |
| | الجلسة الثامنة | ١٢.٠٠-٢.٣٠ | التطهير بالكلور | • خصائص الكلور<br>• استخدامات الكلور فى محطات المياه<br>• نظرية عمل الكلور |
| اليوم الرابع | جلسة الإفتتاح | ٩ – ٩.٣٠ | إستقبال و تسجيل المشاركين فى الدورة | |
| | الجلسة الأولى | ٩.٣٠-١١.٣٠ | المياه المصادر والخصائص | • المياه الجوفية<br>• المياه السطحية |

| اليوم | الجلسة | التوقيت | الموضوع | المحتوى |
|---|---|---|---|---|
| | استراحة | ١١.٣٠-١٢.٠٠ | استـــــــــــــراحـــــة | |
| | الجلسة الثانية | ١٢.٠٠-٢.٣٠ | أنظمة معالجة مياه الشرب | • المعالجة التقليدية |
| | الجلسة الثالثة | ٢.٣٠-٤.٠٠ | محطات الترشيح السريع والمباشر | • المميزات والعيوب |
| اليوم الخامس | الجلسة الرابعة | ٩.٠٠-١١.٣٠ | عمليات معالجة مياه الشرب | • الترويب |
| | استراحة | ١١.٣٠-١٢.٠٠ | استـــــــــــــراحـــــة | |
| | الجلسة الخامسة | ١٢.٠٠-٢.٣٠ | تابع عمليات معالجة مياه الشرب | • التنديف |

| اليوم | الجلسة | التوقيت | الموضوع | المحتوى |
|---|---|---|---|---|
| | الجلسة السادسة | ٢.٣٠-٤.٠٠ | عمليات معالجة مياه الشرب | • الترويق |
| اليوم السادس | الجلسة السابعة | ٩.٠٠-١١.٣٠ | عمليات معالجة مياه الشرب | • الترشيح |
| | استراحة | ١١.٣٠-١٢.٠٠ | استـــــــــــراحـــة | |
| | الجلسة الثامنة | ١٢.٠٠-٢.٣٠ | عمليات معالجة مياه الشرب | • غسيل المرشحات , مشاكل المرشحات |
| | الجلسة التاسعة | ٢.٣٠-٤.٠٠ | زيارة محطة مياه الشرب و عمليات التقييم | • المعايير المستخدمة فى تقييم مياه الصرف ودلالتها وطرق الاستفادة منها فى محطات معلجة مياة الشرب |

# الطرق التقليدية فى معالجة مياه الشرب



# مقدمة فى معالجة مياه الشرب

# مقدمة فى معالجة مياه الشرب

يرجع اهتمام الإنسان بنوعية الماء الذي يشربه إلى أكثر من خمسة آلاف عام . ونظرا للمعرفة المحدودة في تلك العصور بالأمراض ومسبباتها فقد كان الاهتمام محصور في لون المياه وطعمها ورائحتها فقط . وقد استخدمت لهذا الغرض ـ وبشكل محدود خلال فترات تاريخية متباعدة ـ بعض عمليات المعالجة مثل الغليان والترشيح والترسيب وإضافة بعض الأملاح ثم شهد القرنان الثامن والتاسع عشر الميلاديان الكثير من المحاولات الجادة في دول أوربا وروسيا للنهوض بتقنية معالجة المياه حيث أنشئت لأول مرة في التاريخ محطات لمعالجة المياه على مستوى المدن .





## مقدمة فى معالجة مياه الشرب

ومع التطور الشامل للعلوم والتقنية منذ بداية هذا القرن واكتشاف العلاقة بين مياه الشرب وبعض الأمراض السائدة فقد حدث تطور سريع في مجال تقنيات المعالجة حيث أضيفت العديد من العمليات التي تهدف بشكل عام إلى الوصول بالمياه إلى درجة عالية من النقاء ، بحيث تكون خالية من العكر وعديمة اللون والطعم والرائحة ومأمونة من النواحي الكيمائية والحيوية.

## مقدمة فى معالجة مياه الشرب
## طرق المعالجة التقليدية

تختلف عمليات معالجة مياه الشرب باختلاف مصادر تلك المياه ونوعيتها والمواصفات الموضوعة لها . ويجب الإشارة الى أن التغير المستمر لمواصفات المياه يؤدي أيضا في كثير من الأحيان إلى تغير في عمليات المعالجة . حيث أن المواصفات يتم تحديثها دوما نتيجة التغير المستمر للحد الأعلى لتركيز بعض محتويات المياه وإضافة محتويات جديدة إلى قائمة الموصفات . ويأتي ذلك نتيجة للعديد من العوامل مثل

- التطور في تقنيات تحليل المياه وتقنيات المعالجة.
- اكتشاف محتويات جديدة لم تكن موجودة في المياه التقليدية أو كانت موجودة ولكن لم يتم الانتباه إلى وجودها أو مدى معرفة خطورتها في السابق.
- اكتشاف بعض المشكلات التي تسببها بعض المحتويات الموجودة أصلا في الماء أو التي نتجت عن بعض عمليات المعالجة التقليدية . هذا ويمكن تناول عمليات المعالجة التقليدية المستخدمة للمياه استنادا إلى مصادرها السطحية والجوفية



## مقدمة فى معالجة مياه الشرب
## معالجة المياه السطحية :

تحتوي المياه السطحية ( المياه الجارية على السطح ) على نسبة قليلة من الأملاح مقارنة بالمياه الجوفية التي تحتوي على نسب عالية منها ، وهي بذلك بعد مياه يسرة ( غير عسرة ) حيث تهدف عمليات معالجتها بصورة عامة إلى إزالة المواد العالقة التي تسبب ارتفاعا في العكر وتغيرا في اللون والرائحة ، وعليه يمكن القول أن معظم طرق معالجة هذا النوع من المياه اقتصر على عمليات الترسيب والترشيح والتطهير . وتتكون المواد العالقة من مواد عضوية وطينية ، كما يحتوي على بعض الكائنات الدقيقة مثل الطحالب والبكتيريا . ونظرا لصغر حجم هذه المكونات وكبر مساحتها السطحية مقارنة بوزنها فإنها تبقي معلقة في الماء ولا تترسب .

## مقدمة فى معالجة مياه الشرب
## معالجة المياه السطحية :

إضافة إلى ذلك فإن خوصها السطحية والكيميائية باستخدام عمليات الترويب الطريقة الرئيسية لمعالجة المياه السطحية ، حيث تستخدم بعض المواد الكيمائية لتقوم بإخلال اتزان المواد العالقة وتهيئة الظروف الملائمة لترسيبها وإزالتها من أحواض الترسيب .ويتبع عملية الترسيب عملية ترشيح باستخدام مرشحات رملية لإزالة ما تبقى من الرواسب ، ومن المكروبات المشهورة كبريتات الألمنيوم وكلوريد الحد يديك ، وهناك بعض المكروبات المساعدة مثل بعض البوليمرات العضوية والبنتونايت والسليكا المنشطة. ويمكن أيضا استخدام الكربون المنشط لإزالة العديد من المركبات العضوية التي تسبب تغيرا في طعم ورائحة المياه . تتبع عمليتي الترسيب والترشيح عملية التطهير التي تسبق إرسال تلك المياه إلى المستهلك .



## مقدمة فى معالجة مياه الشرب
## معالجة المياه الجوفية:

تعد مياه الآبار من أنقى مصادر المياه الطبيعية التي يعتمد عليها الكثير من سكان العالم . إلا أن بعض مياه الآبار وخصوصا العميقة منها قد تحتاج ألى عمليات معالجة متقدمة وباهظة التكاليف قد تخرج عن نطاق المعالجة هي إضافة الكلور لتطهير المياه ثم ضخها الى شبكة التوزيع ، إذ تعد عملية التطهير كعملية وحيدة لمعالجة مياه بعض الآبار النقية جدا والتي تفي بجميع مواصفات المياه ، الا أن هذه النوعية من المياه هي الأقل وجودا في الوقت الحاضر ، لذلك فأنه إضافة لعملية التطهير فان غالبية المياه الجوفية تحتاج الى معالجة فيزيائية وكيمائية إما لإزالة بعض الغازات الذائبة مثل ثاني أكسيد الكربون وكبريتيد الهيدروجين ، أو لإزالة بعض المعادن مثل الحديد والمغنيز والمعادن المسببة لعسر الماء، وتتم إزالة الغازات الذائبة باستخدام . عملية التهوية والتي تقوم أيضا بإزالة جزء من الحديد والمنجنيز عن طريق الأكسدة ، وقد يكون الغرض من التهوية مجرد كما يحدث لبعض مياه الآبار العميقة التي تكون حرارتها عالية مما يستدعي تبريدها حفاظا على كفاءة عمليات المعالجة الأخرى . أما إزالة معادن الحديد والمنغنيز فتتم بكفاءة في عمليات الأكسدة الكيمائية باستخدام الكلور أو برمنجنات البوتاسيوم .

**مقدمة فى معالجة مياه الشرب**
**معالجة المياه الجوفية:**

ان الطابع العام لمعالجة المياه الجوفية هو إزالة العسر بطريقة الترسيب ، ويتكون عسر الماء بصورة رئيسة من مركبات الكالسيوم والماغنسيوم الذائبة في الماء . ويأتي الاهتمام بعسر الماء نتيجة لتأثيره السلبي على فاعلية الصابون ومواد التنظيف الأخرى ، بإضافة الى تكوين بعض الرواسب في الغلايات وأنابيب نقل المياه. وفيما يلي استعراض موجز للعمليات المختلفة المياه الجوفية في هذا النوع من المحطات . .





**مقدمة فى معالجة مياه الشرب**
**معالجة المياه الجوفية:**

**أ - التيسير ( إزالة العسر ) بالترسيب**

تعني عملية التيسير أو إزالة العسر للمياه ( water softening) إزالة مركبات عنصري الكالسيوم والماغنسيوم المسببة للعسر عن طريق الترسيب الكيمائي . وتتم هذه العملية في محطات المياه بإضافة الجير المطفأ ( هيدروكسيد الكالسيوم ) إلى الماء بكميات محدودة حيث تحدث تفاعلات كيمائية معينة تتشكل عنها رواسب من كربونات الكالسيوم و هيدروكسيد الماغنسيوم . وقد يتم اللجوء في كثير من الأحيان الى إضافة رماد الصودا (كربونات الصوديوم ) مع الجير للتعامل مع بعض صور العسر . وتشمل عملية التيسير على حوض صغير الحجم نسبيا تتم فيه إضافة المواد الكيمائية حيث تخلط مع الماء الداخل خلطا سريعا لتوزيعها في الماء بانتظام ، ثم ينقل الماء الى حوض كبير الحجم ليبقي فيه زمنا كافيا لإكمال التفاعلات الكيمائية وتكوين الرواسب حيث يخلط الماء في هذه الحالة خلطا بطيا يكفي فقط لتجميع والتصادق حبيبات الرواسب وتهيئتها للترسيب في المرحلة التالية ،

## مقدمة فى معالجة مياه الشرب
## معالجة المياه الجوفية:

### ب ـ الترسيب

تعد عملية الترسيب من أوائل العمليات التي استخدامها الإنسان في معالجة المياه . وتستخدم هذه العملية لإزالة المواد العالقة والقابلة للترسيب أو لإزالة الرواسب الناتجة عن عمليات المعالجة الكيمائية مثل التيسير والترويب . وتعتمد المرسبات في أبسط صورها على فعل الجاذبية حيث تزال الرواسب تحت تأثير وزنها .تتكون المرسبات

غالبا من أحواض خرسانية دائرية أو مستطيلة الشكل تحتوي على مدخل ومخرج للماه يتم تصميميها بطريقة ملائمة لإزالة أكبر كمية ممكنة من الرواسب ، حيث تؤخذ في الاعتبار الخواص الهيدروليكية لحركة الماء داخل الخوض . ومن الملامح الرئيسة لحوض الترسيب احتوائه على نظام لجمع الرواسب ( الحمأة ) وجرفها إلى بيارة في قاع الحوض حيث يتم سحبها والتخلص منها بواسطة مضخات خاصة .

## مقدمة فى معالجة مياه الشرب
## معالجة المياه الجوفية:

### ج ـ الموازنة ( إعادة الكربنة ):

نظرا لأن المياه الناتجة هن عملية التيسير تكون في الغالب مشبعة برواسب كربونات الكالسيوم ، وحيث أن جزءا من هذه الرواسب يتبقى في الماء بعد مروره بأحواض الترسيب فإنه من المحتمل أن يترسب بعضها على المرشحات أو في شبكات التوزيع مما يؤدي إلى انسداد أو الحد من كفاءة المرشحات الشبكات . لذلك فإن عملية التيسير لضمان عدم حدوث تلك الأضرار . ومن عمليات الموازنة الأكثر استخداما في التطبيق التقليدية هي إضافة غاز ثاني أكسيد الكربون بكميات محددة بهدف تحويل ما تبقى من كربونات الكالسيوم الى صورة البيكربونات الذائبة ..

## د ـ الترشيح :

هو العملية التي يتم فيها إزالة المواد العالقة ( العكارة ) . وذلك بإمرار الماء خلال وسط مسامي مثل الرمل وهذه العملية تحدث بصوره طبيعية في طبقات الأرض عندما تتسرب مياه الأنهار الى باطن الأرض . لذلك تكون نسبة العكر قليلة جدا أو معدومة في المياه الجوفية مقارنة بالمياه السطحية ( الأنهار والبحيرات وأحواض تجميع مياه الأمطار ) التي تحتوي على نسب عالية من العكر .

تستخدم عملية الترشيح أيضا في إزالة الرواسب المتبقية بعد عمليات الترسيب في عمليات المعالجة الكيمائية مثل الترسيب والترويب .

تعد إزالة المواد العالقة من مياه الشرب ضرورية لحماية الصحة العامة من ناحية ولمنع حدوث مشاكل تشغيلية في شبكة التوزيع من الناحية الأخرى . فقد تعمل هذه المواد على حماية الأحياء الدقيقة من أثر المادة المطهرة ، كما أنها قد تتفاعل كيمائيا مع المادة المطهرة كما أنها قد تتفاعل كيمائيا مع المادة المطهرة مما يقلل من نسبة فاعليتها على الأحياء الدقيقة ، وقد تترسب المواد العالقة في بعض أجزاء شبكة التوزيع مما قد يتسبب في نمو البكتريا وتغير رائحة المياه وطعمها ولونها.تتم عملية الترشيح داخل المرشح الذي يتكون من ثلاث أجزاء رئيسة وهي : صندوق المرشح والتصريف السفلي ووسط الترشيح . يمثل صندوق المرشح البناء الذي يحوي وسط الترشيح ونظام التصريف السفلي ، ويبني صندوق المرشح في العادة من الخرسانة المسلحة ، كما توجد في قاعة ـ الذي يتكون من أنابيب وقنوات مثقبة ـ طبقة

## مقدمة فى معالجة مياه الشرب
## معالجة المياه الجوفية:

من الحصى المدرج لمنع خروج حبيبات الرمل من خلال الثقوب . والغرض من نظام التصريف السفلي تجميع المياه المرشحة وتوزيع مياه الغسيل عند إجراء عملية الغسيل للمرشح . أما وسط الترشيح فهو عبارة عن طبقة من رمل السيليكون ، وحديثا أمكن الاستفادة من الفحم المجروش ورمل الجارنت . عند مرور المياه خلال وسط الترشيح تلتصق المواد العالقة في بجدران حبيبات الوسط ، ومع استمرار عملية الترشيح تضيق فجوات الوسط للمياه بحيث يصبح المرشح قليل الكفاءة وعند ذلك يجب إيقاف عملية الترشيح وغسل المرشح لتنظيف الفجوات من الرواسب يتم في عملية الغسيل ضخ ماء نظيف بضغط عال من أسفل المرشح عبر نظام التصريف السفلي ينتج عنه تمدد الوسط وتحرك الحبيبات واصطدم بعضها مع البعض ، وبذلك يتم تنظيفها مما علق بها من رواسب . وتندفع هذه الرواسب مع مياه الغسيل التي تتجمع في قنوات خاصة موضوعة في أعلى صندوق المرشح ، وتنقل الى المكان الذي يتم فية معالجة مخلفات المحطة وتستمر عملية الغسيل هذه لفترة قصيرة من الزمن (٥ – ١٠ دقائق) بعدها يكون المرشح جاهزا للعمل .





## مقدمة فى معالجة مياه الشرب
## معالجة المياه الجوفية:

### هـ التطهير :

هو العملية المستخدمة لقتل الكائنات الحية الدقيقة المسببة للأمراض (الجراثيم )، وتتم هذه العملية باستخدام الحرارة ( التسخين ) أو الأشعة فوق البنفسجية أو المواد الكيميائية مثل البروم أو اليود أو الأوزون أو الكلور بتركيزات لا تضر بالإنسان أو الحيوان . وتعد طريقة التسخين الى درجة الغليان أولى الطرق المستخدمة في التطهير ولاتزال أفضلها في حمالات الطوارئ عندما تكون كمية المياه قليلة ، لكنها عير مناسبة عندما تكون كمية المياه كبيره كما في محطات المعالجة نظرا لارتفاع تكلفتها . أما استخدام الأشعة فوق البنفسجية والمعالجة بالبروم واليود فتعد طرقا مكلفة . هذا وقد انتشر استخدام الأوزون والكلور في تطهير مياه الشرب ، حيث راج استخدام الأوزون في أوربا والكلور في أمريكا . وفي الآونة الأخيرة اتجهت كثير من المحطات في الولايات المتحدة الأمريكية الى استخدام الأوزون بالرغم من عدم ثباته كيمائيا وارتفاع تكلفته مقارنة بالكلور، وذلك لظهور بعض الآثار السلبية الصحية

**مقدمة فى معالجة مياه الشرب**

**معالجة المياه الجوفية:**

لاستخدام الكلور ( الكلورة ) في تطهير مياه الشرب يتفاعل الكلور مع الماء مكونا حامض الهيبوكلوروز وأيونات الهيبوكلورايت ثم يتفاعل جزء من حامض الهيبوكلوروز مع الأمونيا الموجودة في الماء مكونا أمنيات الكلور( الكلور المتحد المتبقي) ويطلق على ما تبقى من حامض الهيبوكلوروز وأيونات الهيبوكلورايت الكلور الحر المتبقي وهذه المركبات ( الكلور الحر والكلور المتحد )هي التي تقوم بتطهير الماء وقتل الجراثيم الموجودة به ، ولذلك تلجا كثير من محطات المعالجة الى إضافة الكلور بنسب تكفي للحصول على كلور حر متبقي يضمن تطهير الماء الخارج من المحطة بكفاءة عالية ، بل في الغالب تكون كمية الكلور المضاف كافية لتأمين كمية محدود من الكلور الحر المتقي في شبكة توزيع المياه ، وذلك لتطهير المياه من أي كائنات دقيقة قد تدخل في الشبكة .





# الطرق التقليدية فى معالجة مياه الشرب

# المياه الجوفيـــــة





المياه الجوفية هى المياه التى تتجمع فى باطن الأرض من تسرب مياه الأمطار إليها ثم تظهر ثانية بشكل ينابيع أو عيون طبيعية أو يعمل على استخراجها بواسطة الآبار أو الطلمبات و مصدر المياه الجوفية هو مياه الأمطار التى تكون المياه السطحية التى يتسرب جزء منها فى باطن الأرض مكونا المياه تحت الأرضية ويغوص فى الأرض ويكون المياه الغائرة كما هو الحال فى وادى النيل والدلتا وذلك بمرور مياه النهر فى طبقات مسامية (permeable ) يسهل مرور المياه فيها ، ويستمر تسرب المياه حتى تصل الى الطبقة السفلى المشبعة بالمياه (saturated zone) ومستوى المياه فى هذه المنطقة يطلق علية المستوى المائى ويكون المنسوب الأعلى لهذه المياه ثابتاً ويعرف بالمنسوب الثابت للمياه الغائرة (Permanent W.T ) وينحدر هذا المنسوب فى اتجاه سير المياه .

ويتحكم فى مياه هذه المنطقة التركيب الجيولوجى للطبقات ونوع المواد المكونة لهذا الطبقات . فعندما تجرى المياه فى طبقات مسامية ويسهل مرورها دون أن يعوقها عائق فإن هذه المياه تكون حرة فى جريانها ويطلق عليها اسم المياه الحرة ( Free Water )
أما إذا مرت المياه بين طبقتين يصعب مرور المياه بها فإنها تكون مقيدة بهذا الممر وتكون تحت ضغط بحكمة الطرف الأعلى للطبقة الطينية العليا وتسمى بالمياه المقيدة ( Confined Water ) هذا وتعلو الطبقة المشبعة بالماء منطقة قليلة التشبع ( Unsaturated ) وهى تتأثر عادة بمياه المرشح ومنسوبها الأعلى غير ثابت ويعرف بالمنسوب المتغير للمياه الغائرة فى الأرض ( Fluctuating Water Table )



## المياه الجوفية فى مصر

تعتبر المياه الجوفية مصدرا رئيسيا لكثير من موارد المياه العامة. ومصدر المياه الجوفية فى مصر هو مياه الأمطار التى تسقط فى بلاد الحبشة و السودان التى تكون نهر النيل . وعند سقوط الأمطار يتسرب جزء منها داخل الطبقات تحت السطحية (Subsurface ) مكوناً المياه جوفية. وهذه المياه تتكون من منطقتين الأولى طبقة غير مشبعة وهى تعلو الطبقة المشبعة ومياه هذه المنطقة غالبا ما تتأثر بالمياه السطحية والأنهار والقنوات ويتراوح مستواها بين ٦- ١٠ أمتار من سطح الأرض . وقد يصل هذا المستوى الى سطح الأرض فى أوقات الفيضان مسببا تكون برك فى الأماكن المنخفضة ، ومنسوب المياه لهذه المنطقة غير ثابت ويسمى المنسوب المتغير أما الطبقة المشبعة فتمتد تحت الطبقة غير المشبعة وقد سبق الكلام عنها . ويمكن تقسيم المناطق من حيث الانتفاع بالمياه الغائرة فى الأرض الى أربع أقسام:-

١. وادى النيل والدلتا
٢. الصحراء الغربية
٣. الصحراء الشرقية وسيناء
٤. المنطقة الساحلية للبحر الأبيض المتوسط

## أولاً – وادى النيل والدلتا :

وهو عبارة عن مجرى متسع محفور فى الصخور الجيرية وهذا المجرى قد حفره النهر لنفسه فى الصخور فى العصور الماضية ثم ملأه برواسبه المتنوعة من الحصى والرمال والطين وغيرها ، ويختلف اتساع هذا المجرى فى الأماكن المختلفة من الوادى وكذا يختلف عمقه .

ومصدر المياه التى تملأ مسام طبقات رواسبه هى من مياهه ، وجزء من المياه التى تصل الى هذه الطبقات عن طريق وديان الصحارى المصرية كوادى قنا والوادى الأسيوطى وغيرهما أو عن طريق الحجر الرملى النوبى ( Nubian Sand Stone ) الذى يظهر على جانبى الوادى من حدود السودان الى جنوب إسنا والرواسب النهرية لوادى النيل والدلتا عبارة عن مستودع عظيم للمياه التى تصل إليها عن طريق النهر نفسه وهذه المياه تكون كتلاً هائلة متصلة. ومن ذلك يتضح أنه لا توجد فى وادى النيل والدلتا المياه المقيدة تقيدا مطلقا أى التى تنحصر بين طبقتين غير مسامتين التى ينشأ عنها الآبار الارتوازية التى ترتفع مياهها على سطح الأرض وتتدفق منها بسبب الضغط الواقع عليها . كما يتضح أن تسمية الآبار العميقة الموجودة بوادى النيل والدلتا بالمياه الارتوازية غير صحيحة بالمرة ، ويجب عدم الاستمرار فى هذه التسمية والاستعاضة عنها بتعبير ( مياه عميقة تحت التربة ) كما يستعاض عن كلمة أبار ارتوازية بكلمة أبار عميقة .



## ثانيا الصحراء الغربية :

إن المصدر الرئيسي للمياه فى الصحراء الغربية هو الحجر الرملي النوبي ، وهو مشبع بمياه الأمطار التى سقطت علية فى أواسط السودان حيث يظهر هناك على سطح الأرض . وبحكم انحدار هذا التكوين الجيولوجي الى اسفل كلما اتجهنا شمالا ، كانت المياه بداخلة تحت ضغط الطبقات الجيرية التى تعلوه فى الصحراء الغربية ، فإذا ما وجدت هذه المياه طريقها الى سطح الأرض ( على هيئة عيون أو أبار ارتوازية ) سالت وأصبحت موردا هاما لمياه الشرب والرى فى الواحات . وقد ذكرنا هنا كلمة أبار ارتوازية لأن الآبار التى تكون فى هذه المنطقة هى الوحيدة بهذه التسمية لأن مياهها تكون محصورة بين طبقتين وتخرج تحت ضغط . وهناك مصدران أخران للمياه فى هذه المنطقة ، إحداهما النيل عند تقابلة مع الحجر الرملى النوبى الذى يسحب من ماء النيل حينئذ ، والأخر مياه الأمطار على الساحل الشمالى غربى الدلتا – ومن أمثلة الأولى مياه أبار وادى النطرون والأديرة واستراحة طريق إسكندرية الصحراوى .

وقد لوحظ أن منسوب المياه فى الصحراء الغربية قد انخفض فى النصف القرن الماضى حوالى ١٠ امتار فى الواحة الداخلية وحوالى ٥ أمتار فى الواحة الخارجية ويعزى ذلك لكثرة عدد الأبار يكون فى واحتى الخارجة والداخلة ، وأقلها فى واحة الفرافرة –أما فى واحتى البحرية وسيوة فهى بين وبين . أما نوع المياه فقد وجد أن اصلحها وأعذابها فى الواحات الخارجة والداخلة والبحرية ويلى ذلك المياه فى واحة الفرافرة –

## ثالثا الصحراء الشرقية :-

مصدر الماء الرئيسى هو الأمطار ، وهى تسقط على الجبال الشرقية المحاذية للبحر الأحمر وتسيل منها الى الأودية حيث تختزل فى الرمل الحصى.

منطقة سيناء – أهم مصدر للمياه فى هذة المنطقة هى الأمطار . وهى تتجمع فى الوديان مثل وادى العريش ، فيران ، الطور وقد ينتج من هذه المياه عيون ، كما فى منطقتين ( قوس القسيمة والجديرات ) حيث تحفر بيارات لخزن المياه المنسابة منها . وقد اتخذت فى بعض المناطق التدابير اللازمة لخزن مياه السيول وذلك ببناء سدود مثل سد العريش ، غير أنها وجد أن هذة السدود كثيرا ما تمتلىء بالرواسب الجروفة بالسيول أو تتحطم أمامها وتفضل البيارات ذات السدود الأرضية فى هذا الشق .





## رابعا – السهل الساحلى الشمالى :-

ومصدر المياه فيها هو الأمطار التى تسقط على الكذبان الرملية ، فتكون طبقة من المياه العذبة الطافية فوق المياه الراشحة من البحر . ويمكن الحصول على هذة المياه بعمل أبار ضحلة . وتحتوى المياه على قدر كبير من ( الكبريتات الذائبة ) وتزداد ملوحتها كلما اتجهنا غربا فى ساحل سيناء نحو بحيرة المنزلة – أما فى وادى العريش ورفح فهى أعذب من ذلك ولكنها بوجة عام أكثر ملوحة من نظيرتها فى الأبار الضحلة على السهل الساحلى غرب الاسكندرية . وهذا الأخير يحتوى على أبار قديمة ضحلة تسمى الأبار الرومانية يصل قاعها الى طبقة المياه العزبة الطافية ويحفر الأهالى هناك أبارا أخرى وبيارات لخزن مياه الأمطار ويوجد من هذة ما يقرب من الألف بيارة بين الاسكندرية والسلوم ..

## المناطق التى يتعذر الحصول فيها على مياه صالحة للشرب فى وادى النيل

هناك مناطق فى دلتا النيل لا يتيسر الحصول فيها على مياه جوفية صالحة للشرب نظرا لزيادة الأملاح الذائبة بها وهذة المناطق هى :

١. منطقة الفيوم وذلك لوجود طبقات الصخور الجيرية وهى ذات سمك كبير مما يتعذر معة دق أبار عميقة للوصول الى المياه الغائرة كما أن طبقات الرمل والطين التى تعلو هذة الطبقة الجيرية لا يزيد سمكها على بضعة أمتار وهى تحتوى فقط على مياه الصرف المالحة ولذا لا توجد آبار مياه صالحة.
٢. منطقة شمال الدلتا الصحراوية :- وذلك لقربها من البحر الأبيض المتوسط والبحيرات الشمالية وفى هذة المنطقة نجد أن مصدر مياه الشرب هو مياه الأمطار التى تكون طبقة رفيعة لا يزيد سمكها على متر واحد تطفو على المياه المالحة التى تشبع باطن الأرض فى هذة المنطقة .
٣. منطقة قناة السويس وذلك لقربها من قناة السويس والبحيرات المرة وتحتوى طبقاتها على مياه قد تزيد ملوحتها على ملوحة مياه البحر بسبب التبخر وتركيز الأملاح بها
٤. بعض المناطق فى وادى النيل التى تتاخم الصحراء كمناطق الشرقية والبحيرة وبعض المناطق المتفرقة فى الوجة القبلى .



## التكوين الطبيعى للمياه الجوفية

لطبيعة تكوين الارض فى أى جهة أثرا كبيرا ليس فى كمية الماء الجوفية التى يمكن الحصول عليها فحسب بل فى نوع هذة المياه وخواصها الكيماوية والبكتريولوجية . ولتفسير ذلك نذكر أن مياه الأمطار عندما تسقط على سطح الارض فانها تحمل بعض المواد التى تسبب عكارتها ( Turbidity ) كما تلتقط بعض المواد التحللة ( Decomposed ) التى تحوى عددا كبيرا من البكتريا . وعندما تغوص هذة المياه السطحية فى باطن الارض تقوم طبقات الارض الرملية بحجز المواد العالقة بالمياه ومن ضمنها البكتريا . ولذلك فان المياه المستخرجة من الآبار العميقة تكون غالبا رائقة خالية من العكارة . وهذة العملية الطبيعية للترشيح لها أثر كبير فى حجز البكتريا والتقليل منها ويقل عدد من البكتريا كلما زاد تسرب المياه فى الأرض فى طبقات رملية . وقد قام كابر هل ( Cabrhel ) سنة ١٩٠٦ بعمل تحاليل وجد منها أن عدد البكتريا فى السنتيمتر المكعب من المياه السطحية قد بلغ عدة ملايين ، ووصل هذا العدد الى بضع مئات على عمق نصف متر من سطح الارض ، ومائة فى كثير من العينات التى أخذت على عمق زيادة عن متر . أما الآبار العميقة فقد وجد أن مياهها تحوى عددا قليلا من البكتريا العادية ( أقل من ١٠٠ ) وهذه الابحاث تؤيدها التحاليل التى يقوم بها قسم المياه بوزارة الصحة لعينات المياه المختلفة التى ترد له يوميا من أنحاء الجمهورية .

أما من الوجهة الكيماوية فاننا نجد أن المياه الجوفية تختلف فى طبيعتها عن المياه السطحية . فنظرا لأن تربة الارض وصخورها تحوى أملاحا قابلة للذوبان ، فان المياه تذيب أثناء تسربها فى باطن الارض نسبة كبيرة من هذة الاملاح مما يسبب زيادتها فى المياه الجوفية عنها فى المياه السطحية . فبينما نجد أن الأملاح الذائبة فى مياه النيل تتراوح من ١٥٠ – ٢٥٠ جزء فى المليون نجد أن الاملاح فى الآبار الجيدة تبلغ حوالى ٢٥٠ الى ٥٠٠ جزء فى المليون وفى بعض الآبار أقل جودة من ٥٠٠ الى ٨٠٠ جزء فى المليون وفى الباقى من ٨٠٠ – ١٠٠٠ وهذا هو الحد الأقصى الذى تسمح به وزارة الصحة فى المياه الصالحة للشرب واذا زادت الاملاح عن ١٠٠٠ يعتبر غير صالح .

ويمكن أن نلخص خواص المياه الجوفية فى الأتى :-



١. أنها رائقة نتيجة لعملية الترشيح التى تحدث لها أثناء تسربها فى باطن الارض . وقد يجدث أن تكون المياه المستخرجة من البئر رائقة ولكنها تتعكر بعد أستخراجها بفترة بسيطة وهذا يرجع الى ترسب الحديد والمنجنيز وكربونات الكالسيوم والمغنسيوم نتيجة فقدان الغازات التى تجعل هذة المواد فى حالة ذائبة ( عادة ثانى أكسيد الكربون )
٢. الرائحة – ليس للمياه الجوفية فى العادة رائحة ، ولكن بعضها قد يكون محملا بالهيدروجين المكبرت ويكسب المياه رائحة وطعما خاصين ، كما أن الآبار الملوثة تكون للمياه المستخرجة منها رائحة كريهة وغير مستحبة .
٣. الطعم – يتأثر طعم المياه الجوفية بما يكون زائدا فيها من الغازات والأملاح فالمياه التى بها نسبة عالية من الحديد والمنجنيز يكون لها طعما قابضا. كم أن المياه ذات النسبة العالية من الأملاح خاصة كلوريد الصوديوم يمكن تمييز طعمها بالمذاق . والى جانب ذلك فان عسر الماء يمكن تميزة بالمذاق خاصا اذا كانت كمية أملاح الماغنسيوم كبيرة
٤. المياه الجوفية غالبا باردة فى الصيف دافئة فى الشتاء .
٥. والى جانب ذلك قد تحوى المياه الجوفية على مواد عضوية ووجود هذة المواد العضوية فى المياه أمر غير مرغوب فية لأنة دليل على وجود مصدر للتلوث – والمواد العضوية التى توجد فى التربة الأرضية والتى تتسرب مع المياه الأرضية على نوعين أساسين ، كربونية وأزوتيه ، والمواد العضوية الكربونية ، يمكن أن تتأكسد الى ثانى أكسيد الكربون أمام المواد العضوية الأزوتيه فتتأكسد الى نتريت ونترات .

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## تلوث المياه الجوفية

مما لاشك فية أن المياه الجوفية عرضة للتلوث . ويختلف مدى هذا التلوث حسب نوع المصدر وحسب حالة البئر ونوع طبقات الأرض . كما أن مدى تسرب التلوث فى باطن الأرض انما يتوقف على نوعة وقد ثارت عدة أراء حول مدى تأثر البئر بمصادر التلوث القريبة منة وأختلفت الآراء حول مدى تسرب هذا التلوث . واننا نرى أن لكل مورد ظروفة الخاصة حسب حالتة وحسب طبيعة المصدر وحسب طبقات الأرض كما ذكرنا وهذة كلها عوامل يجب أن تؤخذ فى الحسبان عند تقرير صلاحية المورد أو عدم صلاحيتة . فقد يظهر الفحص الكيماوى والبكتريولوجى لمرود ما ، عدم صلاحيتة لوجود ما يشير الى تلوث المياه فى حين يكون البئر بعيدا كل البعد عن مصادر التلوث ثم يتضح ذلك فعلا فى كثير من عمليات المياه الجوفية التى عوينت بمعرفة قسم المياه نظرا لتكرار عدم صلاحية العينات المأخوذة منها .



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ويمكن أن نذكر بصفة عامة أن التلوث البكتيري يتسرب إلي مسافة حوالي ١.٥ متراً والتلوث الكيماوي لمسافة ٢.٥متراً من قاع يؤرة التلوث كالمجارير الغير صماء وغيرها بشرط أن يكون ذلك التسرب تحت منسوب مياه الرشح في تربة ناعمة مغمورة دائماً بالمجاري السائلة . كما أن البكتريا تتسرب مع تيار التلوث السائل لاجانبياً ولا ضده . كما أن سير التلوث يزداد في حالة الأرض ذات الحبيبات الكبيرة نوعاً .

ويمكن القول أن التلوث سواء أكان بكتيريا أو كيماوياً يتسرب مع تيار المياه في الارض إلي أسفل ولكنه ينحرف في اتجاهه عند مقابلته تيار المياه الجوفية

وإذا ما قرب منسوب مياه الرشح من سطح الارض تعرضت المياه للتلوث ببكتريا القولون كما أن قدرة التربة علي حجز البكتريا تتوقف علي حجم الحبيبات فاذا كبرت حجماً زاد تغلغل البكتريا فيها والعكس كما أن البكتريا تمكت مدة أطول في الأرض الخشنة عنها في الأرض الناعمة بعد وصولها اليها ، وحينئذ قد تتكاثر تكاثراً يفوق في أثره مدي التلوث السطحي الأول .

أما عن مدي سيري التلوث مع تيار المياه الجوفية فانه بناء علي عدة مشاهدات نذكر أن البكتريا القولونية تتسرب مع المياه الجوفية في اتجاهها الي مسافة ٧ متر قدماً أما تتسرب المواد الكمياوية الذاتية ( النوشادر والازوتيت والازوتات ) فيكون الي مدي أطول وبسرعة أكبر وتدل التقارير علي امكان تسربها الي مسافة تتراوح بين بضعة أقدام الي عدة أميال . وتشير بعض التجارب التي أجريت أن سير التلوث البكتيري لا يزيد عن ٥ أمتار أما التلوث الكيميائي فقد يصل الي ١٠٠ متر .



## تلوث المياه الجوفية في وادي النيل والدلتا :

من المعلوم أن الرواسب النيليه تتكون من طبقات مسامية غير منتظمة شأن جميع الرواسب النهرية وتحمل هذه الطبقات المياه الغائرة التي تسير في مجموعها من الجنوب الي الشمال في طريقها الي البحر وهو المنفذ الطبيعي لها ويمكن تلخيص أنواع هذه الطبقات فيما يلي:-

١. الطبقة العليا تتكون من رواسب نهرية بسمك حوالي ٦ الي ١٠ متراً يمكن للمياه أن تتسرب خلالها ، وهي تختلف في طبيعتها من طبقات طينيه غير مسامية الي طينيه رملية .
٢. يلي هذه الطبقة طبقة أخري من الرمل والرمل الطيني المشبع بالماء بسمك حوالي ١٠الي ١٥ متر . وهي طبقة ذات مسام دقيقة جداً بحيث يصعب سحب المياه منها بالطلمبات بكميات وافره وهي ذات مقدرة كبيرة علي حجز وترشيح البكتريا.

٣. ويلي الطبقة الثانية طبقة ثالثة من الرمل الخشن والزلط التي يمكن سحب المياه بسهولة بواسطة الأبار العميقة .

٤. ومن المعلوم أن منسوب المياه الجوفية علي عمق حوالي ١٠ متراً ويتغير هذا المنسوب صعوداً وهبوطاً تبعاً لمنسوب مياه النيل وقت الفيضان والتحاريق ، وبذلك تنشأ عمليتي ترشيح وتجفيف علي نطاق واسع واذا أمكن حماية الأبار في وادي النيل والدلتا من التلوث السطحي أمتنع تلوثها من مصادر التلوث كالمجارير التي يبلغ عمقها حوالي ١٠ متراً وذلك اذا بلغت هذه الأبار عمقاً يتراوح بين ١٣ و ١٥ متراً أي وصلت الي الطبقة المشبعة بالمياه وتحت منسوبها البكتريا اليها وخاصة مع قصر مدة التخزين في طبقات الأرض الذي هو عامل من عوامل اهلاك البكتريا .

وبناءً علي ذلك فانه لضمان عدم تلوث الأبار يجب أن يكون سحب المياه من أعماق تزيد عن العشرة أمتار وهو متوسط عمق المنسوب للمياه الجوفية .

وهناك عوامل أخري يجب أن ندخلها في اعتبارنا عند دراسة احتمال التلوث كنوع المصدر وعمقه وتصميم البئر ومسامية الأرض ومدي انخفاض مخروط السحب وتأثيره علي المنسوب المائي





## الطريقة الصحيحة للحصول على المياه الجوفية :-

أن الطريقة الصناعية الوحيدة للحصول علي المياه الجوفية هي بواسطة الأبار وهناك اشتراطات يجب أن تتوافر فيه مكان البئر وفي اقامته وتعقيمه وحمايته من التلوث .

وكل مال أو جهد يبذلان في اقامة بئر دون مراعاة الشروط الصحية إنما هو مال ضائع وجهداً بلا نتيجة وهذا يستلزم تعاون المختصين الصحيين والكيماويين والمهندسين الصحيين وغيرهم .

## إختيار موقع البئر :

من أهم مايجب مراعاته عند إقامة بئر جديد إختيار موقعه بحيث يكون هذا الموقع ملائماً للغرض الذي أقيم من أجله .ومن الإعتبارات التي يجب مراعاتها في هذا الصدد مايأتي :-

١. أن يكون البئر في جنوب المدينة أو القرية وإذا كان المورد سيتطلب إقامة عدد من الأبار فيجب تصميمها بحيث تكون ممتدة علي خط متجه من الشرق إلي الغرب لا من الجنوب إلي الشمال حتي لا تتأثر مياه البئر الشمالي من السحب من البئر الجنوبي .
٢. ألا يكون البئر في أرض منخفضة حتي لا يكون عرضه لتجمع المياه السطحية حوله ، كما في حالات الأمطار أو الفيضانات .
٣. أن يكون بعيداً عن المساكن وفي وسط مساحة واسعة يمكن إقامة منشآت العملية بها ويكون من اليسر عمل حرم للمأخذ لمنع التلوث .
٤. ألا يكون في منطقة الأرض بها غير سليمة أو بها شقوق .
٥. أن يكفل ( وهو المهم ) إمداد المنطقة بكميات المياه اللازمة للسكان للشرب والأغراض المنزلية وكذا الأغراض التجارية والصناعية وأغراض البلدية .



## حماية البئر من التلوث :-

يجب عند اختيار مكان للبئر أن يكون بعيداً عن مصادر التلوث التي تأثر عليها أو التي يحتمل أن تؤثر عليها في المستقبل فيجب أن يكون البئر بعيداً عن مصادر التلوث بمسافة كافية وهذا البعد يختلف كما ذكرنا فيما سبق بإختلاف الظروف المحلية كما يتوقف علي عدة عوامل منها :-

١. نوع وموقع مصدر التلوث
٢. نوع البئر وتصميمه
٣. مدرج المنسوب المائي
٤. مسامية الأرض وتركيبها
٥. مدي انخفاض مخروط المياه الجوفية أثناء سحب المياه من البئر

فمن ناحية نوع وموقع مصدر التلوث نجد أن التلوث الناتج من صرف متخلفات المراحيض يفوق في أثره التلوث الناتج من منزل قريب للبئر ، كما أن نوع البئر وتصميمه له علاقة وثيقة بالتلوث . فالبئر الذي يسحب مياهه من المنطقة المشبعة ( كالأبار العميقة ) يكون أقل عرضة للتلوث من بئر يسحب مياهه من المنطقة الغير مشبعة .

ومدرج المنسوب المائي وحركة المياه الجوفية لهما دخل كبير في سير التلوث الأرضي ، وكلما تقدينا المنطقة ذات المنسوب المتغير كلما كان ذلك أضمن و أفضل .

أما مسامية الأرض وتركيبها فتؤثر تأثيراً كبيراً علي مدي سير التلوث أو حجزه وقد سبق أن ذكرنا أنه كلما ذادت مسامية الأرض كلما أدي ذلك إلي سهولة تسرب المياه دون أن يتم ترشيحها الترشيح الكافي وبالتالي حجز ما بها من البكتريا .

أما عن مخروط السحب فلتفسيره نذكر أنه عند تشغيل الطلمبة لسحب المياه من البئر يترتب عليها انخفاض منسوب الماء في البئر ويؤدي ذلك إلي انخفاض مستوي المياه حول البئر وهذا ما يطلق عليه ( Cone of Depression ) أي مخروط السحب . ويترتب علي تكون هذا المخروط انحدار المياه نحو البئر من جميع الجهات مما يغير من الإتجاه الطبيعي للمياه الجوفية ، ويسمح للتلوث السطحي الواقع في منطقة السحب بالتسرب بسرعة إلي البئر وكلما ذاد انحدار مخروط السحب نحو المأخذ كلما ذاد احتمال وصول المياه الملوثة إلي البئر . وكلما ذاد تشغيل الطلمبة الساحبة للمياه ، كلما ذاد هذا الإنحدار – وعلي ذلك فإحتمال التلوث يزداد بازدياد السحب في حالة زيادة استهلاك المياه . ويمكن القول عموماً أنه لحماية البئر من التلوث يجب أن تتوافر فيه الشروط الأتية :-

١. أن يكون عمقه ٢٠ متر علي الأقل .
٢. أن يكون بعده عن مصدر التلوث ٢٠ متراً علي الأقل وذلك بالطلمبات الميكانيكية . أما في الطلمبات اليدوية فيمكن أن يكون البعد ١٠ متراً من مصدر التلوث بشرط توفر العمق وبشرط أخذ عينات دورية للفحص البكتريولوجي والكيماوي فإذا ثبت تلوث المياه أوقف إستعمالها .



## المصادر االتى تؤدى الى تلوث الأبار :

هناك عدة مصادر يمكن ان يترتب عليها تلوث . يمكن اعتبار المصادر الأتية كمصادر يترتب عليها تلوث الأبار اذا لم تستوف الابعاد والمسافات الواجبة

خزانات المراحيض الغير صماء – بيارات الصرف – أكوام السباخ – حظائر الماشية –المصارف – الجبانات – البرك والمستنقعات – المياه الراكدة حول المأخذ أو بالقرب منة – منازل الفلاحين – مزارع المجارى – الاراضى الزراعية التى تسمد بالسماد العضوى – العيوب التى قد توجد فى التصميم واقامة الآبار ( شروخ فى المواسير ، قدمها ، تأكلها ) ترنشات الصرف .

ويجب على الذين يعملون فى الميدان الصحى ان يراعوا عند اختبار مورد المياه ، أن يتأكدوا من عدم وجود أحد هذه المصادر أو غيرها على مسافة ٢٠ مترا فى حالة العمليات الميكانيكية أو ١٠ متر فى حالة الطلمبات اليدوية . وأن يذكر فى أورنيك ارسال العينة جميع البيانات عن مصدر التلوث واحتمال تأثيرة على مياه البئر سواء وقت أخذ العينة أو فى المستقبل

## الآبـــــار

الآبار هى الوسيلة الصناعية التى نحصل بها على المياه الجوفية ، وقد عرفت الآبار منذ قديم الازل الا أنها لم تكن متوافرة الشروط الصحية سواء فى أقامتها أو صيانتها . وكانت غالبا من النوع المكشوف الذى تستخرج منة المياه بواسطة الدلو ، وهذا النوع من الآبار المكشوفة مازال موجودا فى بعض القرى الا أن عددا كبيرا منها قد تم ردمه وحلت محله مشروعات المياه الكبرى . وان كان هناك بعض القرى المحرومة من المياه الصالحة للشرب مازال بها عدد من هذة الآبار .



## الاحتياطات الواجب اتخاذها اثناء دق الأبار أو اقامتها

يجب اتخاذ كافة الاحتياطات التي تكفل عدم تلوث المياه أثناء دق أو اصلاحة . وفيما يلي نورد هذه الاحتياطات :

يجب أن يحرص الأشخاص والعمال القائمين بدق البئر علي عدم تلويث مجاوراته لمسافة ١٠٠ متر حوله . كما يجب عدم استخدام مواسير أو أدوات تكون قد تلوثت بالشحم أو السماد. وأن يتخذوا جميع الاحتياطات الكافيله بدرء التلوث الخارجي عن البئر .

يجب غسل جميع المواسير والقاسون المستعمل بمحلول مركز من هيبوكلوريت الصوديوم ويطلق عليه ( جير كلور) قبل انزالها في الأرض.

إذا استعمل الزلط في إقامة البئر فيجب غسله في محلول مركز من هيبوكلوريت الصوديوم ( جير الكلور ). والنسبة المستعمله في هذه الحالة هي نصف كيلو من هيبو كلوريت الصوديوم إلي ٦ لتر من الماء

بعد الانتهاء من دق البئر وإنزال المواسير المجلفنه يجب تعقيم المياه قبل أخذ العينات وذلك بإستعمال مادة هيبو كلوريت الصوديوم قوة ٢٥% وستذكر فيما يلي طريقة تعقيم البئر .

## طريقة تعقيم الأبار

١. يمزج ٥٧ جم من مسحوق جير الكلور قوة ٢٥% في٠ ٢ لتر ماء لكل ٤٠٠ لتر من المياه داخل الماسورة ، ويتم تحضير هذه المادة بأن تضاف كمية بسيطة من الماء إلي كمية جير الكلور ويقلب بهدوء حتي نحصل علي عجينة ، ثم نضيف إليها كمية الماء ( ٢٠لتر لكل ٤٠٠ لتر من الماء في الماسورة ) ويقلب السائل بهدوء لمدة ١٠ إلي ١٥ دقيقة ويترك ليرسب ويأخذ السائل الذي يحتوي علي الكلور الفعال لإستعمالة في التعقيم .

٢. يضاف السائل الناتج من رقم (١) إلي المياه الموجودة في الماسورة ويترك لمدة ٤٨ ساعة وفي خلال هذه الفترة يبدء في إدارة الطلمبه حركة بسيطة تتلوها حركة عكسية دون إخراج الماء وهذه العملية تتيح مزج المطهر بالماء كما يتيح لهذه المياه تعقيم جدران الماسورة الداخلية أثناء صعودها عند إدارة الطلمبه أو نزولها عند إيقاف الإدارة بحركة عكسية ويجب إجراء هذه العملية عدة مرات .



٣. يبدأ بعد مضي الـ ٤٨ ساعة المتقدمة في إدارة الطلمبة إدارة تامة لتفريغ محتويات الماسورة حتي يزول كل أثر الكلور في الماء ( تكون فترة الإدارة ٢٤ساعة )

٤. إذا تتطلب الأمر أخذ عينات فتكون علي الأساس الأتي :-

٥. العينة الأولي بعد ٢٤ ساعة إدارة مستمرة من انتهاء فترة التعقيم وتفريغ الماسورة

٦. العينة الثانية بعد إدارة ٢٤ ساعة مستمرة من العينة الأولي .

٧. العينة الثالثة بعد إدارة الطلمبة ٢٤ ساعة بعد العينة الثانية .

## حماية موارد المياه الجوفية

من أهم الأعباء التي تقع علي عاتق القائمين بالرقابة علي موارد المياه وصيانتها العمل علي اتخاذ الاحتياطات الكافيله بحماية المورد من التلوث وهذه الاحتياطات متنوعة يمكن أن نلخص بنودها الأساسية في الأتي :-

١. بعد رفع القاسون وانزال المواسير يجب عمل تغطية من الأسمنت أو أي مادة صماء حول الماسورة بحيث تشمل هذه التغطية الجزء غير المخرم منها ويتم عمل هذه التغطية بالأسمنت بطريقة الضغط وتشترط بعض الاشتراطات في الأسمنت المستعمل وفي طريقة تحضيرة بحيث يكون طبقة متماسكة حول الماسورة ولا يكون به شقوق تسمح للمياه السطحية بالتسرب إلي المأخذ .

٢. يستحسن عدم وضع الماكينة فوق ماسورة الغز مباشرةً بل يتم تركيب الماكينة علي مداد من ماسورة الغز وذلك لإمكان إصلاح أي عيب يطرأ علي الماسورة دون حاجة إلي رفع الماكينة .



٣. يجب عمل دكة أسمنت حول ماسورة المأخذ لمسافة دائرة نصف قطرها ٥ أمتار من كل ناحية وتكون هذه الدكه مائله من البئر إلي خارجه بشكل هرمي وذلك لحماية المأخذ من تسرب المياه السطحية حوله .

٤. يجب عمل حرم للبئر لمسافة ٢٠ متر من جميع الجهات وأن يتم عمل سور متين علي هذه المسافة مع إزالة جميع مصادر التلوث التي تكون موجودة في هذا الحرم

٥. يجب عدم وضع مواسير شبكة توزيع المياه أو أي محابس بجانب مواسير المجاري ، بل تكون شبكة مواسير المياه مستقله عنها . مع مراعاة نظافة هذه المواسير قبل وبعد وضعها .

٦. يجب عمل طريقة صرف صحية للمياه المستعمله في تبريد الماكينات أو المياه الفائضة من الصهريج أو المتخلفة من غسيل الصهريج بحيث تصرف هذه المياه بعيداً عن المأخذ .

٧. إذا وجدت مراحيض لموظيفي العمليه أو دورات المياه فيجب تصميم طريقة صرف صحية لها مع مراعاة أن تكون بيارات الصرف أو الترنشات والخزانات علي مسافة لا تقل عن ٢٠ متر من أبار العملية .

## خزانات وصهاريج المياه

إن إقامة عمليات المياه العامة يتطلب عمل خزان للمياه وتتوقف ساعة الخزان وحجمة علي نوع العملية وقوتها والإستهلاك اليومي وإعتبارات فنية أخري وتتم إقامة هذا الخزان بحيث يكون أعلي من منزل في المدينة حتي تصل المياه إلي الأدوار العليا من المنازل . ولما كانت المياه قد تبقي في هذا الصهاريج مدة طويلة وبإستمرار الإستعمال يتعرض الخزان للتلوث ويؤدي ذلك بالتالي الي تلوث المياه المستعمله . ولذا وجب علي القائمين علي عمليات المياه والمشرفين مولاة التأكد من حالة هذه الخزانات وعدم تلوثها . ومن الأمور الهامه التي يجب مراعاتها في هذه الخزانات ما يلي :-



١. يجب أن يكون موقع الخزان في مكان نظيف وان يكون الوصول اليه سهلاً
٢. يجب أن يغطي بغطاء محكم لمنع وصول الأتربه إليه .
٣. أن تعمل له ماسورة للمياه الفائضة بحجم مناسب حتي يمكن إبطار تشغيل الماكينة إذا ما إمتلأ الخزان .
٤. يجب أن يصمم الخزان بحيث تسحب المياه من أعلي القاع بقليل ليمكن للرواسب أن تتجمع في هذه المسافة . ويكون من السهل إزالتها عند غسله وتعقيمه .
٥. إذا كان الخزان من الصاج فيجب أن يتم دهانه ببويه مانعة من الصدأ وأن يتم تحليل هذه البويه بمعامل وزارة الصحة قبل إستعمالها للتأكد من خلوها من المواد السامة وإلي جانب هذه الإستراطات العامة فيجب ضرورة تعقيم الخزان بمادة جير الكلور مرتين في العام ما لم يطرأ طارئ يستدعي التعقيم ( كما في حالة الأوبئه ) وفيما يلي طريقة تعقيم الخزانات :-

## طريقة تعقيم الخزانات

١. يجب إعداد جميع مواد النظافة والتعقيم ( جير الكلور ) قبل البدء في العملية
٢. يفرغ الخزان من المياه مع الإبقاء علي كمية منها بإرتفاع ٤٠: ٥٠ سم تسعمل في تنظيف الخزان ، أرضيته وجدرانه .
٣. العمال الذين يقومون بعملية التنظيف يجب أن يلبسوا أحذيه من المطاط ذات الرقبه الطويله بعد وضعها لبضع دقائق في محلول مخفف من كلورور الجير ( واحد في الألف تقريباً ) أي بنسبة ملعقه كبيرة في جردل ماء سعته حول ١٠لتر .
٤. بعد إزالة جميع المواد الملتصقة بأرضية وجدران الخزان بالفرش ، تطرد مياه الغسيل بالعادم ثم تمرر كمية مياه جديده وتطرد ، وتكرر هذه العملية حتي لا تبقي أية رواسب بأرضية الخزان.
٥. يعقم الخزان بإستعمال مسحوق جير الكلور الذي قوته ٢٥% ( علي الأقل ) وذلك باستحلابه بمعدل ١٠ جرام لكل متر مكعب حسب سعة الخزن فمثلاً لتعقيم خزان سعته ١٠٠ متر يستحلب كيلو جرام في جردل علي دفع ويضاف إلي مياهه أثناء ملئه ويترك ساعتين ، ثم يفرغ ويبدء في ملئه للإستعمال .)
٦. في غير فترات غسيل وتعقيم الخزان يمكن عمل طرد من آن لأخر (Periodic flushing ) للتخلص من أية رواسب موجودة بأرضية الخزان وذلك بإدخال كميات قليله من المياه وهو فارغ ثم طردها للعادم دون وصول أي شخص بداخله وفي هذه الحالة لا



## مزايا المياه الجوفيه

١. للمياه الجوفيه يمكن أن نلخصها في الأتي : -
٢. أن الأمر لا يتطلب فيها إقامة منشأت بنايه كبيرة للمياه لأن الأرض تعتبر مخزن للمياه .
٣. أنها أقل عرضه للتلوث خاصتاً تلك التي نحصل عليها بواسطة الأبار العميقه لأن قدرة الأرض علي ترشيح المياه تكون كافية لصلاحية المياه المسحوبه من البئر إلا في بعض الحالات الأتية:-
٤. إذا كانت طبيعة الأرض مشبعه بالمواد العضوية
٥. إذا كانت هناك خزانات لصرف مجاري أو أي مصادر تلوث أخري قريبة من البئر
٦. ج- إذا كان هناك شقوق في سطح الأرض تتسرب منها المياه السطحية إلي المأخذ
٧. ٣- لا يتطلب الأمر إجراء عملية ترسيب أو ترشيح للمياه الجوفية ولاكن قد يحتاج الأمر إلي تعقيمها بالكلور كاإجراء وقائي .
٨. ٤-إن أقامة بئر أو عدة أبار للمياه يحتاج إلي مساحة قليله من الأرض كما أن النفقات أقل . ولذلك فهي مفضله في حالة مشروعات المياه التي تهدف إلي تغذية أكبر عدد ممكن بأقل النفقات .

## مساوئ المياه الجوفيه

١. إحتمال عدم الحصول علي الكميات الكافية من المياه من الأبار الخاصة في المدن الكبيرة التي يزداد فيها الاستهلاك منما يتطلب ضرورة تدبير موارد أخري .
٢. أن المياه غالباً غسره ولابد من عمل تيسير لها إذا ذاد هذا العسر كما أن طعمها قد يكون ردئ . وإذا كانت المياه الجوفيه ستسعمل في أغراض الصناعه فإن العسر المؤقت يؤدي إلي أملاح كربونات وكبريتات الكالسيوم وهي تؤدي إلي خفض درجة الحرارة في الغلايات وقد يتسبب عنها إنفجار هذه الغليات
٣. وجود الحديد والمنجنيز بنسبة كبيرة يتطلب ضرورة إزالته حتي لا يكون للماء شكل أو طعم غير مرغوب فيه وذلك علاوه علي تأثير الحديد في الأقمشه .





# الطرق التقليدية فى معالجة مياه الشرب

# المعالجة التقليدية لمياه الشرب



## الهدف من عمليات التنقية :-

يقصد بالتنقية التخلص من كل او بعض المواد الغريبة سواء كانت ذائبة او عالقة او غروية . حيث ان المياه السطحية معرضة لعوامل كثيرة تؤدى الى تلوثها فتصبح غير صالحة للاستعمال الا بعد تنقيتها

## عمليات المعالجة

- تمر المياه الخام بعدة عمليات لمعالجتها لتكون صالحة للشرب وهى:

١. المعالجة اولية وتنقسم إلى:

التصفية

- الـ تطهـ ير الـمـ بدئـ ى

المعالجة الكيماوية المبدئية

الترسيب المبدئى (اختيارى)

٢ - . المعالجة الرئيسية وهى تلى المعالجة الاولية وتنقسم إلى:

التر ويب

التنديف

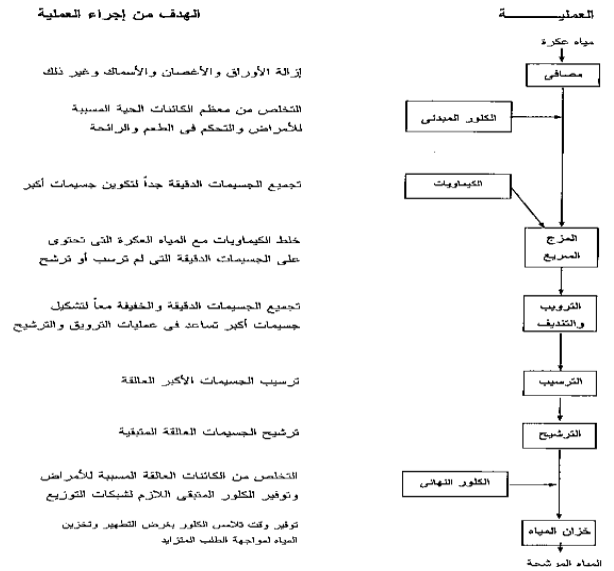
الترسيب

الترشيح

التطهير/ التعقيم

التخزين

الضخ








# الترسيب

## مقدمة

الغرض من عملية الترسيب هو العمل على ترسيب اكبر نسبة ممكنة من المواد العالقة و التى لها ثقل اكبر من دفع الماء و ذلك عن طريق تركها ترسب تحت تاثير وزنها . و عملية الترسيب اما ان تكون طبيعية ، اى تحت تاثير وزن المواد العالقة بدون اضافة اى مواد و تسمى هذه العملية بالترسيب الطبيعى. او تكون باضافة مواد مساعدة كيماوية للماء لتساعد على تجميع المواد الغير قابلة للترسيب و تسمى هذه العملية بالترسيب بمساعدة المروبات

** العوامل المؤثرة على كفاءة عمليات الترسيب منها:-

١. تركيز المواد العالقة
٢. شكل و حجم و كثافة المواد العالقة
٣. درجة حرارة و لزوجة المياه
٤. مدة بقاء الماء فى الاحوض

## نظرية الترسيب الطبيعى

يحدث الترسيب الطبيعى نتيجة هبوط الحبيبات منفردة، و يحدث التؤسيب تحت تاثير وزن الحبيبة فقط و يحدث مقاوة اثناء هبوط الحبيبات المنفردة نتيجة لزوجة المياه و حيث ان قانون نيوتن يوضح انه اذا توازنت القوى المؤثرة على الحبيبات .لا يحدث عجلة تسارع و لكن تحدث سرعة هبوط ثابتة

و تعتمد نظرية الترسيب على تخفيض سرعة المياه الداخلة الى احواض الترسيب الى ادنى درجة بحيث يتم ايقاف الحركة الافقية للحبيبات العالقة و العمل فقط بالحركة الافقية مما يساعد على سرعة الترسيب قبل خروج المياه من احواض الترسيب و دخولها الى المرشحات و التى تقوم بازالة المتبقى من الحبيبات التنى لم يتم اذالتها اثناء عملية الترسيب

.

و تقاوم الحبيبات الغير قابلة للترسيب عملية الترسيب لسببين اساسيين هما :-

١. حجم الحبيبات نفسها
٢. القوى الطبيعية بين الجسيمات





## أ . حجم الجسيمات

تنقسم الجسيمات من حيث الحجم الى

١. مواد معلقة
٢. مواد غروية
٣. مواد ذائبة

قابلية ترسيب الجسيمات فى المياه

| قطر الحبيبات (مم) | النوع | زمن الترسيب* | القابلية للترسيب |
|---|---|---|---|
| ١٠ | حصى | ١ ثانية | قابلة |
| ١ | رمل | ١٠ ثوان | قابلة |
| ٠,١ | رمل ناعم | ٢ دقيقة | قابلة |
| ٠,٠١ | طمى / طحالب | ٢ ساعة | غير قابلة |
| ٠,٠٠١ | بكتريا | ٨ أيام | غير قابلة |
| ٠,٠٠٠٢ | مواد غروية | سنتين | غير قابلة |
| ٠,٠٠٠٠٢ | فيروسات | ٢٠ سنة | غير قابلة |

## أولاً : المواد الصلبة المعلقة:

هى جسيمات ينقلها معه الفعل الطبيعى للمياه الجارية وهى أكبر من ( ٠.٠٠١ مم) و الجسيمات التى لا تتجاوز( ٠.٠١ مم ) حجماً لا تترسب بسرعة وهى تسمى فى عمليات معالجة المياه "مواد غير قابلة للترسيب" أما الأكبر حجماً (أكبر من ٠.٠١< مم) فتوصف بانها قابلة للترسيب ، حيث أنها تترسب تلقائيا إلى القاع أو حوض الترسيب فى غضون أربع ساعات



## ثانيا: المواد الصلبة الغروية:

هى جسيمات الطمى الدقيق والبكتريا والجسيمات المسببة للون والفيروسات وهذه الغرويات لا تترسب تلقائيا في مدة زمنية معقولة كما هو موضح بالجدول وبالرغم من أنه لا يمكن رؤيتها بالعين المجردة إلا ان تاثيرها قد يشاهد عند مرورها خلال عمليات المعالجة إذا لم تتروب تندف كما يجب

## ثالثاً : المواد الصلبة المذابة:

هى أى مواد عضوية أو غير عضوية "الاملاح والكيماويات التى من أصل نباتى أو حيوانى أو غازات) يمكن ذوبانها فى المياه، والمادة الصلبة المذابة هى جسيمات صغيرة جداً (فى حجم الجزئ) ولا يمكن رؤيتها بالعين المجردة وغالبا تكون المعادن المذابة فى الماء مواد صلبة غير قابلة للترسب، وتسبب مشكلات للصحة العامة ومشكلات أخرى مثل الطعم أو اللون أو الرائحة وما لم تحول إلى مواد مترسبة بواسطة وسيلة كيميائية أو فزيائية فلا يمكن إزالتها من الماء

ومن الاسباب الرئيسية التى تجعل الجسيمات غير قابلة للترسيب هو صغر حجمها والذى يمكن توضيحة بالمثال التالى:-

## مثال:

من المتوقع أن يترسب حجم من رمل خشن على شكل مكعب طول ضلعه ١ مم بحيث يهبط نحو ٠.٣ متر كل ٣ ثوان والآن لنفرض أن حبيبة الرمل قد طحنت إلى عدة جسيمات أصغر، كل منها مكعب طول ضلعه ٠.٠٠٠١ مم (هذا الطحن يماثل قوى التعرية الطبيعية) فإن الوزن الاجمالي لكل الجسيمات المطحونة يساوى وزن الحبيبة الاصلية للرمل الخشن.

وبمقارنة المساحة السطحية فى الحالتين نجد انها قد زادت من ١ مم ٢ وهى مساحة رأس دبوس كبير إلى ٦ م٢ وهذه الزيادة فى المساحة السطحية تسبب زيادة هائلة فى قوى المقاومة والتى تقاوم الترسيب الطبيعى التلقائى وطبقاً للجدول فبدلاً من ان ترسب تلك الجسيمات الدقيقة إلى عمق٣ . ٠ متر فى ٣ ثوان فإنها تستغرق الآن نحو ٢ ٠ سنه لترسب إلى نفس العمق لذلك فمن اللازم لتعجيل الترسيب أن يتم تجميع الجسيمات الدقيقة معاً لتكون جسيمات أكبر يمكن ترسيبها بسهولة وسرعة.



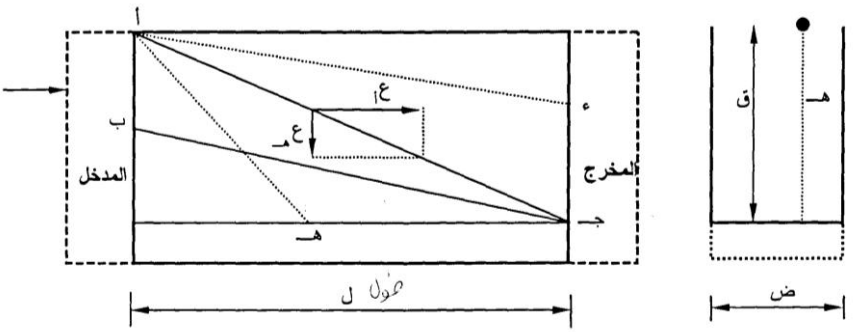
## ب . القوى الطبيعة

تحمل عادة الجسيمات الموجودة فى المياه شحنات كهربية سالبة وكذلك فإنه توجد هناك قوة تنافر بين أى جسمين متماثلين فى الشحنة. وفى عملية معالجة المياه تسمى قوة التنافر الكهربية جهد زيتا Zp وهى قادرة على إبقاء الجسيمات الغروية الصغيرة جدا متباعدة بعضها عن بعض ومعلقة فى الماء.

وتوجد قوة أخرى هى قوة فان درفال بين جميع الجسيمات فى الطبيعة وهى تعمل على جذب أى جسمين معا. وقوة الجذب هذه تعمل فى اتجاه مضاد لجهد زيتا وطالما كان جهد زيتا أقوى من قوة فان درفال فإن هذه الجسيمات تظل معلقه.

ومن الناحية النظرية البحتة فإنه يمكن توضيح عملية الترسيب كما هو موضح فى الشكل . كمدخل لتفهم أسس تصميم عملية معالجة المياه. ذلك على أساس أن المواد العالقة متجانسة التوزيع فى الماء. وبفرض أن سرعة المياه بما فيها من مواد عالقة فى الاتجاه الأفقى (ع) وأن تصرف المياه يساوى( ك) وأن عرض الحوض (ض) وعمقه ( ق) وتكون مساحة مقطعه (ضXق) وتكون سرعة المياه الافقية = معدل التدفق/ مساحة المقطع.

$$ع = \frac{ك}{ض \times ق}$$








وتكون هى أيضاً السرعة الافقية للمواد العالقة بمختلف أحجامها
ع أ سرعة الأجسام العالقة فى الاتجاه الأفقى
ع هـ سرعة الأجسام العالقة فى الاتجاه الراسى (سرعة الهبوط)
ق عمق الحوض
ض عرض الحوض
ح المساحة السطحية لحوض الترسيب
أء مسار المواد التى لا ترسب وتخرج من الحوض
ب جـ مسار المواد التى لا يرسب منها إلا ما كانت نقطة دخوله بالقرب من القاع
أ هـ مسار الأجسام ذات الكثافة والحجم الأكبر والتى ترسب بسرعة
أما سرعة المواد العالقة فى الاتجاه الرأسى ع أ فتكون مختلفة حسب حجم الجزيئات وكثافتها. ومن الشكل نستنتج أن:

$$\frac{ع_{ا}}{ع_{ـ}} = \frac{ل}{ق}$$

$$\therefore ع_{ـ} = \frac{ق}{ل} \times ع_{ا}$$

وحيث أن

$$ع_{ا} = \frac{ك}{ض \times ق}$$

$$\therefore ع_{ـ} = \frac{ك}{ض \times ق} \times \frac{ق}{ل} = \frac{ك}{ض \times ل}$$

أى أن:

$$\text{سرعة الهبوط الرأسية للحبيبات} = \frac{\text{معدل التدفق (التصرف)}}{\text{المساحة السطحية لحوض الترسيب}}$$





ويتضح من ذلك أن المواد العالقة التى لها سرعة هبوط رأسية مساوية أو أكثر من ع هـ يتم ترسيبها فى الحوض. وعلى العكس لا ترسب المواد إذا كانت سرعتها الراسية أقل من ع هـ .

ومن ذلك نرى أن المساحة السطحية لحوض الترسيب(ض X ل) لها تاثيرمباشر على كفاءة الترسيب. فكلما ذادت المساحة السطحية قلت ع هـ وزادت كفاءة الحوض معدل التدفق.و تسمى ع هـ بمعدك التحيل اسطحى لحوض الترسيب. وتتخذ كأساس من أمس التصميم لهذه الاحواض. وتكون وحدتها متر/ساعة

ومن الشكل يتبين أن المسار أ هـ هو مسار المواد التى لها سرعة هبوط رأسية أقل من ع هـ وهذه لا ترسب فى الحوض وتخرج منه ، ولا يرسب منها إلا ما يدخل منها قرب قاع الحوض والذى يمثله المسار (ب جـ). أما المسار أ هـ فيبين مسار المواد التى لها كثافة وحجم أكبر، وترسب بسرعة لأن سرعتها الرأسية تكون أكبر من ع هـ

## الترسيب بمساعدة المروبات

عندما تبين عدم ملاءمة الترسيب الطبيعى للطلبات المتزايدة على المياه ء وأيضا لعدم جدواه فى ترسيب الحبيبات الدقيقة والخفيفة و خاصة المواد الكلويدية والعوالق الطينية والكائنات الحية الدقيقة والمواد العضوية الناتجة من تحلل النباتات المائية ء وذلك إما لصغر سرعة الترسيب بشكل كبير وبالتالى تحتاج إلى مدة مكث كبيرة .و لأن الجسيمات الموجودة بالماء تحمل شحنة كهربية سالبة وبالتالى يحدث تنافر بينها لتماثل الشحنات فتبقى

الجسيمات متباعدة عن بعضها. لذلك تضاف إلى المياه مواد كيماوية (مروبات) تعمل على تجميع هذه الحبيبات الرفيعة ، كما سبق توضيحه . وبعد عمليتى المزج السريع والمزج البطئ تمر المياه فى أحواض الترويق حيث ترسب الندف المتكونة فى احواض الترويق بما جذبت إلى سطحها من مواد عالقة إلى قاع الحوض.

- ولا تختلف أحواض الترويق فى تصميمها عن أحواض الترسيب الطبيعى، كما أن العوامل المؤثرة على كفاءة الترسيب فيها لاتختلف عن احواض الترسيب الطبيعى ولكن فى وقت أقل جداً ، وبالتالى تكون أحجام هذه الآحواض أصغر كثيراً. وبالرغم من اختلاف أنواع وطرازات احواض الترويق إلا أنها تتفق فى الاسس الرئيسية للتصميم وان اخلفت فى بعض التفاصيل .



## العوامل المؤثرة فى عملية الترسيب

تتاثر عملية الترسيب بعدة عوامل أهمها:-

١. حجم الحبيبات: وطريقة توزيعها، فكلما زاد حجمها ووزنها، ازدادت كفاءة الترسيب .
٢. شكل الحبيبات :- فكلما اقترب شكلها من الشكل الكروى كلما كان ترسيبها اسرع و اكفاء
٣. كثافة الحبيبات: فكلما زادت كثافتها زادت كتلتها بالنسبة لحجمها وزادت سرعة رسوبها وبالتالى كفاءة الترسيب .
٤. درجة حرارة الماء: فكلما ارتفعت درجة حرارته قلت كثافته ولزوجته وبالتالى زادت سرعة رسوب الحبيبات وزادت كفاءة الترسيب .
٥. الشحنة الكهربائية للجسيمات: والتى تكون دائما سالبة الشحنة، وعند معالجة المياه بالشبة موجبة الشحنة ، يحدث تجاذب بين الجسيمات السا لبة والموجبة مما يساعد على ترسيبها وزيادة كفاءة الترسيب .
٦. سرعة سريان الماء فى الحوض: فكلما قلت سرعة الماء ، زادت كفاءة الترسيب، ويفضل ألا تتجاوز السرعة الافقية فى الحوض( ٣٠ سم/ دقيقة)
٧. مدة بقاء الماء فى الحوض : مدة المكث فكلما ذادت المدة ذ ادت جودة الترسيب. ومن النواحى الاقتصادية والعملية أن تكون مدة المكث فى حدود ٣ : ٤ ساعات و حيث أن زيادة المدة أكثر من اللازم لا يزيد من كفاءة الترسيب إلا بنسبة بسيطة.

$$\text{مدة المكث} = \frac{\text{حجم حوض الترسيب}}{\text{معدل التصرف خلال الحوض}}$$

النسبة بين طول وعرض حوض الترسيب فى الأحواض المستطيلة. وذلك لاقلال فرص تكون مناطق راكدة أو ميتة عند زيادة عرض الحوض .





## عملية الترويب و التنديف

تتكون الشوائب الموجودة فى المياه نتيجة التآكل الأرضى الناتج من احتكاك المياه مع قطاع المجرى المائى، وتتكون الشوائب من دقائق صغيرة كما تلتقط المياه مع الاحتكاك ايضا كميات من الاملاح وبقايا النباتات المتحلله كما توجد شوائب أخرى إضافية ناتجة من التلوث الهوائى ومخلفات المصانع و فضلات الحيوانات. ولذلك فالمياه السطحية تتلوث بتاثير الطبيعة و الانسان. وتحتوى المياه على مواد عالقة ومواد ذائبة عضوية(من اصل نباتى اوحيوانى) وكذلك على المواد غير العضوية وكافة الاشكال البيولوحية( كالبكتريا والنباتات المائية والهائمات ) وتسمى هذه المواد" المواد العالقة الصلبة" وتكون قدرأ كبيرأ من مكونات الشوائب. أما الاجزاء الكبيرة الحجم مثل الرمل والطمى الثقيل فيمكن ازالتها من المياه بتركها للترسيب بواسطة الجاذبية الارضية و تسمى هذه المواد بالمواد الصلبة القابلة للترسيب

- و يحدث ترسيب للمواد الصلبة طبيعيا عند تخزين المياه فترة كافية فى الخزانات او فى الانهار و البحيرات ذات السرعة البطيئة . لكن الاجزاء الصغيرة جدأ مثل المواد الغروية والطمى الدقيق والبكتريا التى لا تترسب بسهولة وتحتاج إلى معالجة لتجعل جزيئاته أكبر وذات قابلية للترسيب. وتسمى هذه المواد بالمواد غير القابلة للترسيب أو المواد الغروية .

## الحاجة الى الترويب و التنديف

الغرض من تلك العمليتين هو ازالة المواد الشائبة و خاصة غير القابلة للترسيب الطبيعى و ازالة العكارة من المياه المراد معالجتها

- وتعمل المواد الكيماوية (المروبات) على إزالة المواد الغير قابلة للترسيب بعملية الترويب ونلك بأن تجعل هذه الجزئيات تتجمع مع بعضها البعض لتكون الندف الصغيرة ثم تتجمع الندف الصغيرة مع بعضها البعض لتكوين نفدف كبيرة و ثقيلة قادرة على الترسيب. تخلط المواد الكيماوية بالماء العكر الذى يحتوى على جسيمات دقيقة لا تترسب و لا تترشح بسهولة و تعمل على جعل الجسيمات الدقيقة الشديدة الصغر تتجمع وتتكتل معاً فى صورة أجسام أكبر (ندف) للمساعدة فى إتمام عمليتى الترسيب والترشيح ثم ياتى الترسيب بعد ذلك ليرسب الجسيمات العالقة الكبيرة ثم الترشيح ليحتجز ويستخرج الجسيمات العالقة الباقية.



## و صف عملية الترويب و التنديف

تؤدى عملية التر ويب/ التنديف إلى تعادل أو تقليل جهد زيتا للمواد الصلبة غير القابلة للترسيب بحيث يمكن لقوة فان درفال أن تبدأ فى جذب الجسيمات معاً وعندئذ تتمكن الجسيمات الغير قابلة للترسب من التجمع فى مجموعات صغيرة من الندف الدقيقة . ورغم أن هذه الجسيمات أكبر من الغروية الأصلية إلا أنها ضعيفة التماسك مع بعضما وتكون الندف الدقيقة الغير مرئية للعين المجردة وتظل غير قابلة للترسيب ويودى فعل التقليب البطىء الذى يحدثه التنديف إلى تجميع الجسيمات الندفية معاً لتكوين جسيمات ندفية كبيرة وثقيلة نسبياً يمكن ترسيبها .و ترشيحها بسهولة، والجسيمات الندقية الهلامية القوام تكون مرئية فى العادة وتبدو مثل ندف القطن او الصوف. .

## استخدام المرويات(مواد الترويب)

- وحيث أن معظم الجسيمات غير المرغوبة والمطلوب إزالتها من المياه سالبة الشحنة فبالتالى يجب أن تكون المرويات المستعملة فى معالجة المياه ذات شحنه موجبة ، فالشحنات المرجبة تعادل الشحنات السالبة وتعجل الترويب .
- وتحتوى بعض المرويات على أيونات ذات شحنات موجبة أكثر من البعض الأخر فنجد أن تلك المشتملة على أيونات ثلاثية التكافوء مثل الالمونيوم و الحديد ($Fe^{+++}$, $AL^{+++}$) أشد فعالية بمقدار يتراوح من ٧٠٠ - ١٠٠٠ مرة (كمرويات) من الايونات الأحادية التكافو مكر الصوديوم ($Na^{+}$) وأشد فعالية بمقدار ٥٠- ٦٠ مرة من الأيونات ثانية التكافو ط الكالسيوم ($Ca^{++}$ )
- ومن المواد الشائعة المستخدمة فى عملية الترويب هى كبريتات الالمونيوم و اسمها الدارج الشبة , $AL_2(SO_4)3.nH_2O$و عند ذوبان هذا المروب و تاينه فى الماء فإنه يكون الأيونات ثلاثية التكافو للألومونيوم $Al^{+++}$ وتتراوح نطاق الجرعة من ( ٥ - 100) ملليجم/ لتر .



## أولا: الترويب

### تعريف عملية الترويب

توضح كلمة الترويب التأثير الناتج عندما تضاف مواد كيماوية معينة للمياه الخام المحتوية على مواد بطينة الترسيب ومواد لا تترسب طبيعياً ، ويمكن تعرف الترويب على أنه بداية استجابة الجسيمات بطينة الترسيب فى المياه الخام لعمل ندف دقيقة عند إضافة المروب للمياه مشكلاً سحابة هلامية ويسلزم ذلك عملية خلط سريع لضمان توزيع متماثل للمادة المروبة .

### الخلط السريع

هو مزج المواد المروبة مع المياه الخام والغرض من المزج السريع هو التوزيع المتماثل للمواد الكيماوية المروبة خلال المياه وتحدث هذه العملية فى وقت قصير جداً عدة ثوان لم وتكون النتيجة الأولية لهذه العملية هى تكوين جزيئات صغيرة جدا. فالتلامس الأول للمروب مع المياه هى الفترة الزمنية الأكثر حرجاً فى عملية الترويب بأكملها وذلك لأن تفاعل الترويب يحدث بسرعة، وعليه فمن المهم أن يتلامس المروب والجسيمات الغروية فوراً وبعد إضافة المروب.

## وصف العملية الترويبية من الناحية الكيماوية:

تعتبر العملية الترويبية من العمليات المعقدة وهى عملية فيزيائية وكيميائية تحث بين قلوية المياه (وهى قدرة المياه على معادلة الأحماض) الناتجة من وجود مكونات مثل الكربونات والبيكربونات والهدروكسيدات واليورات والسيليكات والفوسفات بالمياه وبين المادة المروبة.

ويساعد تقليل الرقم الهيروحينى للمياه( ٨ )pH على تهيئة الوسط للشحنة الموجبة وهو المناسب للتفاعل مع الجزيئات و الغرويات السالبة الشحنة مكون أندف غير ذانبة التى تعمل على إزالة الشوائب من المياه.

ويعتبر مدى الرقم الهيروحينى من ( ٥.٥ - ٧.٥ ) مدى مناسب لعملية الترويب وتعطى الشبة أفضل نتائج فى هذا المدى حيث أن المادة المروبة تتفاعل مع جزء من القلوية فى الماء، والقلوية المتبقية بعد التفاعل والموجودة فى المياه تعمل كمنظم تمنع الرقم الهيدروحينى للمياه من التغير وبالتالى تساعد على الترسيب الكامل للمادة المروبة ويمكن تمثيل معادلة التفاعل كالآتى:





كبريتات الألومونيوم (الشبة) + القلوية (بيكربونات الكالسيوم) ←——— هيدروكسيد الألومونيوم (الندف) + كبريتات الكالسيوم + ثانى أكسيد الكربون

وفى حالة عدم وجود القلوية فى المياه يضاف إليها الصودا الكاوية أو الجير المطفأ Ca(OH)2وتكون المعادلة كالتالى :

كبريتات الألومونيوم + الجير ( $Ca(OH)_2$ ) ←——— هيدروكسيد الألومونيوم (الندف) + كبريتات الكالسيوم

وبصفة عامة فإن المشغل لا يستطيع التحكم فى القلوية .و الرقم الهيدروحينى العالى بمصدر المياه ولذلك فمؤشر تقييم نوعية المياه يلعب دوراً رئيسيا فى اختيار نوع المادة المروبة المستخدمة فى معالجة المياه. كما أن الجرعة الزائدة او المنخفضة تؤدى إلى الاقلال من كفاءة عملية الازالة ، ويتم التاكد من إنجاز هذه المهمة بعمل Jar test

## العوامل المؤثرة فى انجاز عملية التر ويب

يوجد عدة عوامل يجب اخذها فى الاعتبار لانجاز عملية التر ويب بكفاءة وهى كالتالى:

**طرق المزج**

يجب مزج المواد العروبة (الشبة) مع المياه بأسرع وقت (عدة ثوان)
حيث أن زمن هذا التفاعل قصير ، ثم تتم عملية التفاعل بعد ذلك بعدة طرق منها

- من نها:
  - المزج الهيدروليكى
  - المزج الميكانيكى
  - المزج بنظام الهواء المضغوط
  - المزج بطريقة الحقن فى طلمبات المياه العكرة

لكى تتم عملية الترويب والتنديف فإنه يجب أن يلامس المروب كل اجزاء المواد العالقة ويحدث ذلك بعملية التقليب السريع



## ثانيا :- التنديف

**تعريف عملية التديف**

- هى عملية مزج بطئ للشبة تعمل على تجميع الحبيبات الصغيرة لتكوين حبيبات أكبر قابلة للترسيب ويستلزم ذلك عمل تقليب بطئ باستخدام الخلاطات الهيدروليكية والميكانيكية لإتاحة فرصة أكبر لعمل تلامس بين الندف الدقيقة لتلتصق ببعضها مشكلة ندف اكبر لترسب ، وتتطلب هذه العملية زمن يتراوح من ١٥ :٢٠ دقيقة .

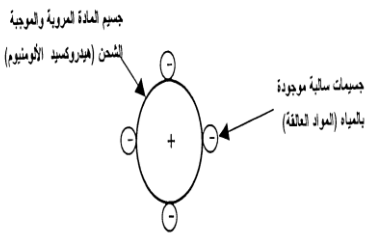
- **وصف العملية**

يتراوح الحجم الامثل للندف من (١ مم - ٣ مم) ويعتمد ذلك على نوع عملية الازالة المستخدمة سواء كان بنظام الترشيح المباشر أو الترشييح التقليدى ويجب الاخذ فى الاعتبار وقت المزج والطريقة الصحيحة للمزج وشكل أحواض المزج. فالمزج السريع لمدة كبيرة يجعل الندف المتكونة تتباعد عن بعضها البعض وبالتالى تتكون ندف ضعيفة ومفككة بعد ان كانت ملتصقة ببعضها.

فالتنديف عملية أبطأ بكثير من الترويب لذلك فإن حوض التنديف يجب أن يكون أكبر نسبياً ونظراً لأن الندف هشة تماماً فلذلك يجب أن يجرى الخلط بطيئاً كما يجب أن تكون سرعة التصرف خلال الحوض بطيئاً بما يكفل عدم تفتيت جسيمات الندف .و تكسيرها وعلى ذلك يجب أن يكون حوض التنديف كبرا بما يكفى. لتهيئة مدة المكث الملائمة.

ويحمل هيدروكسيد الألومونيوم المتكون شحنة كهربائية موجبة، وبالتالى يحدث تجاذب بينه وبين المواد العالقة والغرويات الدقيقة والبكتريا والمواد الغروية والتى تحمل شحنة سالبة ، فيحدث تعادل كهربائى، وتلتصق ببعضها مكونة حبيبات أكبر تسمى ندف، والتى تزداد حجماً وثقلا أثناء تحولها فى الماء نتيجة تماسك الندف الدقيقة مع بعضهاه مكونة ندفا أكبر واثقل ، قابلة للترسيب بسرعة أكبر. وتسمى هذه العملية بالتندف






- و كل جزء فى المليون ( جم/ م٣) من الشبه الصلبةيقلل قلوية الماء بنسة٤٥.٠ جزء فى المليون ويزيد ثانى أكسد الكربون بمقدار ٠.٤ جزء فى المليون

**تأثير إضافة جرعة شبة قدرها ٣٠ جم/م٣**

| م | نوع التحليل | مياه عكرة | مياه مرشحة |
|---|---|---|---|
| ١ | القلوية (ppm) مقاسة ككربونات الكالسيوم | ١٨٠ | ١٦٦ |
| ٢ | ثانى أكسيد الكربون (ملجم/ لتر) | ١,٤ | ١٢,٦ |
| ٣ | الرقم الأيدروجينى pH | ٨,١ | ٧,٥ |
| ٤ | العكارة (NTU) | ١٨ | ٤,٦ |

## العوامل التى تؤثر فى عملية الترويب و التنديف

تتاثر عملية الترويب و التنديف بعولمل مختلفة :-

- تركيز الاس الهيدروجينى (pH) ولكل نوع من المواد المروبة مدى فى الرقم الهيدروجينى. فتتم عملية الترويب بأعلى كفاءة بالنسة للشبة عندما يكون الرقم الهيدروجينى من ٤.٥ إلى ٧.٥ . اما بالنسبة لكبريتات الحديدوز فتحدث عند رقم هيدروجينى أكبر من ٨.٥
- قلوية الماء ،حيث تتم عملية التر ويب اسرع مع القلوية العالية .
- ظروف خلط المادة المروبة، ويفضل أن يكون خلط المادة المروبة بسرعة وبتجاس فى كل حجم المياه الخام .
- نسبة العكارة ، ويفضل ألا تقل عن نسبة معينة ،فإذا كانت قليلة جداً فأحياناً يتم اللجوء إلى إضافة مواد مساعدة لتكوين نواة تتجمع حولها الندف، ويؤدى ذلك إلى الإقلال من المادة المروبة .
- جرعة المادة المروبة ، والمقصود بها عدد جرامات المادة المروبة التى يتم إضافتها للمتر المكعب من الماء الخاما ويعبر عنها بالجرام لكل متر مكعب . و بجزء فى المليون وذلك لاتمام عملية التر ويب والتندف باعلى كفاءة ولتكوين ندف ذات اكبر كثافة. ويفضل تحديد جرعة المادة المروبة عن طريق التجارب العملية باستخدام اختبار تحديد الجرعات (Jar test)



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- **ويتناسب حجم الندف طردياً مع جرعة الشبة المضافة وذلك فى حدود معينة، وبالتالى يلزم تحديد جرعة الشبة التى تحقق تكوين الندف فى حجم رأس الدبوس
- ** ويتم تحديد جرعة الشبة الفعالة معملياً، بواسطة جهاز تحديد الجرعات الشبه (Jar test)





• **تجهيز اعمال الترويب و التنديف**

**مقدمة:-**

- للبدء فى أعمال الترويب والتندف يجب اولاً العناية بتداول وتخزين المواد المروبة ثم بعد ذلك البدء فى تحضير المحاليل الخاصة منها ليتم إضافتها للمياه متى تتم عملية التر ويب والتنديف بكفاءة.

**تخزين الكيماويات**

- تتوفر المروبات فى شكل صلب او مسحوق أو حبيبات او سائل وغالباً ما تكون السوائل اسهل فى الخلط والا ستعمال ويمكن شراء الكيماويات باحجام مختلفة تتر واح من اكياس زنة ١٥ كيلو جرام وأوعية سعة ٢٠ لتر الى سيارات نقل صهريجية زنة ٥٠ طن أو أكثر .
- ويجب دائماً تخزين الكيماويات فى منطقة جافة عند درجة حرارة متوسطة ومنتظمة الى حد ماء حيث أن الرطوبة قد تسب تصلد او تجبس الكيماويات الجافة فمن الواجب تخزينها على طبالى خشبية نقالة للسماح بمرور الهواء أسفل الاكياس ومن المفضل ايضاً تخزين الكيماويات الجافة على أرضية تقبع فوق منطقة تحضير المحاليل حتى يمكن تغذيتها بالجاذبية الى المنطقة الخاصة بالتحضير وعلى أية حال فقد لا يكون ذلك عملياً فى المحطات الصغيرة والمتوسطة الحجم لأن حجم الكيماويات المستعملة قد لايعتبر إستثماراً رئيسياً فى معدات تداول الكيماويات .
- وتشمل المعدات المستعملة فى تداول الكيماويات العربات اليدوية أو الميكانيكية والأوناش العلوية والمصاعد والناقلات الميكانيكية .

## تحضير المحاليل

- يتم عادة تخفيف المرويات فى محطة المعالجة الى محاليل ذات تركيز مناسب قبل إضافتها الى المياه الخام وغالباً ما يكون التركيز ١٠ % صلبة
- وفى المحطات الصغيرة يمكن إجراء ذلك عن طريق تفريغ الأكياس يدوياً أو تفريغ الكيماويات السائلة عن طريق السيفون، أو الطلمبات الى أحواض التخفيف لتحضير المحاليل المغذية. ومن المهم أن يعتنى المشغل بخلط المادة الجافة أو السائلة خلطاً متساوياً ومتجانساً فى الماء، وفى المحطات الكبيرة يخصص مبنى كامل للتحضير الاتوماتيكى لمحاليل الترويب وقد يشمل مثل هذا المبنى على قوادين كبيرة وموازين وناقلات بسيور وأوناش وشبكات من المواسير وطلمبات لمعايرة الكيماويات وصهاريج للتخزين المؤقت .
- ويتكون محلول الشبة من محلول حمضى آكل يتفاعل مع المادة الحاوية له، لذلك يتم إنشاء أحواض تجهيز المحلول من المبانى أو الخرسانة ثم يتم تبطينها بمادة لا تتأثر بالأحماض مثل السيراميك او تدهن بمواد لا تتاثر بالأحماض مثل الإيبوكس والفيبرجلاس اوالألياف الزجاجية



## تحضير المحاليل المخففة من الشبة

- يتم إضافة مأئة كيلو جرام من الشبة الصلبة (شيكارتين زنة الواحدة ٥٠ كجم ) لكل متر مكعب من المياه فى الحوض لتعطى تركيز ١٠ % أما في حالة الشبة السائلة ( ٥٠ %) ذات كثافة ١.٣٢ فيتم التخفيف الى ٢٠ % من وزنها ( ما يعادل ١٠ % صلبة)

مثال : حوض أبعاده ٢ × ٢× ٦ متر (إرتفاع)

سعة حوض التخفيف ٢٤ م$^{٣}$

كثافة الشبة ٥٠% = ١,٣٢

كتلة الشبة فى الحوض كصلبة (١٠٠%) = $\frac{٢٤ \times ١٠٠}{١٠٠٠}$ = ٢,٤٠٠ طن

كتلة الشبة فى الحوض كسائلة (٥٠%) = ٢,٤ × ٢ = ٤,٨ طن

حجم الشبة فى الحوض كسائلة (٥٠%) = $\frac{٤,٨}{١,٣٢}$ = ٣,٦٣ م$^{٣}$

أى يضاف ٣٦٣٦ لتر فى حوض سعة ٢٤ م٣ من الشبة (٥٠%) لتعطى تركيز ١٠% وهو يعادل = ٣٦٣٦ × ١,٣٢ = ٤,٨ طن.

| حجم محلول الشبة م٣ | ارتفاع المياه | حجم المياه م٣ | كمية الشبة/طن | حجم الشبة م٣ |
|---|---|---|---|---|
| ٢٤,٠ | ٥,٠٩١ | ٢٠,٣٦٤ | ٤,٨ | ٣,٦٣٦ |
| ٢٢,٠ | ٤,٦٦٦ | ١٨,٦٦٧ | ٤,٤ | ٣,٣٣٣ |
| ٢٠,٠ | ٤,٢٤٢٥ | ١٦,٩٧٠ | ٤ | ٣,٠٣٠ |
| ١٨,٠ | ٣,٨١٨ | ١٥,٢٧٣ | ٣,٦ | ٢,٧٢٧ |
| ١٦,٠ | ٣,٣٩٤ | ١٣,٥٧٦ | ٣,٢ | ٢,٤٢٤ |
| ١٤,٠ | ٢,٩٦٩ | ١١,٨٧٩ | ٢,٨ | ٢,١٢١ |
| ١٢,٠ | ٢,٥٤٥ | ١٠,١٨٢ | ٢,٤ | ١,٨١٨ |
| ١٠ | ٢,١٢١ | ٨,٤٨٥ | ٢ | ١,٥١٥ |
| ٨,٠ | ١,٦٩٧ | ٦,٧٨٨ | ١,٦ | ١,٢١٢ |
| ٦,٠ | ١,٢٧٢ | ٥,٠٩١ | ١,٢ | ٠,٩٠٩ |
| ٤,٠ | ٠,٨٤٨ | ٣,٣٩٤ | ٠,٨ | ٠,٦٠٦ |

- جدول يوضح كمية وحجم الشبة السائلة اللازمة لتحضير محلول تركيزه يعادل ١٠% صلبة لإرتفاعات مختلفة من الحوض.



## تحضير محاليل من الشبه الصلبة و السائلة

- يجب تخفيف الشبة الى تركيز مناسب قبل إضافتها الى الماه الخام فى أحواض، ويضاف الماء النقى اليها لتحضير المحلول كما يلى :
- **أ. فى حالة الشبة الصلبة(شكاير ٥٠ كجم ):**

بفرض ان سعة حوض الشبة ١٥٤ م٣ (١١ متر طول X٣.٥ عرض ٤ X م إرتفاع) فإنه يتم إسقاط ٣٠٨ شيكارة شبة صلبة فى الحوض للحصول على محلول بتركيز ١٠ % ( يتم اسقاط شيكارتين لكل ١ م٣ من حجم الحوض.

- **ويتم إتباع الخطوات التالية لتجهيز المحلول المطلوب :**

١ . يتم إضافة مياه للحوض من المحبس وعندما يصل منسوب المياه لمستوى ريش الخلط يتم تشغيله لتقليب المحلول.
٢ . يغلق محبس إضافة المياه للحوض عند ملئه.
٣ . استمر فى التقليب حتى يتم إذابة كل الشبة.
٤ . إبلاغ المعمل لأخذ عينة فى اليوم التالى.
٥ . يتم السحب من حوض الشبة حتى يصل الى مستوى عوامة أقل منسوب

## ب . فى حالة الشبة السائلة( تركيز ٥٠ % من الخزان):

للحصول على محلول شبة تركيز ١٠ % يتم ملء الحوض بالمياه لأى إرتفاع نختاره طبقاً للجدول ثم يفتح محبس دخول الشبة السائلة بتركيز ٥٠ % من الخزان بالارتفاع المناظر له من الجدول مع التقليب بواسطة الخلاط للحصول على محلول شبة تركيز ١٠ %





تحضير محلول شبة بتركيز ١٠% باستخدام شكاير الشبة الصلبة

| ارتفاع محلول الشبة ١٠% المطلوب تجهيزه (م) | حجم محلول الشبة ١٠% المطلوب تجهيزه (م٣) | عدد شكاير الشبة • |
|---|---|---|
| ٤,٠٠ | ١٥٤,٠٠ | ٣٠٨,٠٠ |
| ٣,٩٠ | ١٥٠,١٥ | ٣٠٠,٣٠ |
| ٣,٨٠ | ١٤٦,٣٠ | ٢٩٢,٦ |
| ٣,٧٠ | ١٤٢,٤٥ | ٢٨٤,٩ |
| ٣,٦٠ | ١٣٨,٦ | ٢٧٧,٢ |
| ٣,٥٠ | ١٣٤,٧٥ | ٢٦٩,٥ |
| ٣,٤٠ | ١٣٠,٩ | ٢٦١,٨ |
| ٣,٠ | ١٢٧,٠٥ | ٢٥٤,١ |
| ٣,٢ | ١٢٣,٢ | ٢٤٦,٤ |
| ٣,١ | ١١٩,٣٥ | ٢٣٨,٧ |
| ٣,٠ | ١١٥,٥ | ٢٣١,٠٠ |
| ٢,٩٠ | ١١١,٦٥ | ٢٢٣,٣٠ |
| ٢,٨٠ | ١٠٧,٨ | ٢١٥,٦ |
| ٢,٧٠ | ١٠٣,٩٥ | ٢٠٧,٩ |
| ٢,٦ | ١٠٠,١ | ٢٠٠,٢ |
| ٢,٥ | ٩٦,٢٥ | ١٩٢,٥ |
| ٢,٤ | ٩٢,٤ | ١٨٤,٨ |
| ٢,٣ | ٨٨,٥٥ | ١٧٧,١ |
| ٢,٢٠ | ٨٤,٧ | ١٦٩,٤ |
| ٢,١ | ٨٠,٨٥ | ١٦١,٧ |
| ٢,٠ | ٧٧,٠ | ١٥٤,٠ |
| ١,٩٠ | ٧٣,١٥ | ١٤٦,٣ |
| ١,٨ | ٦٩,٣ | ١٣٨,٦ |
| ١,٧ | ٦٥,٤٥ | ١٣٠,٩ |
| ١,٦ | ٦١,٦ | ١٢٣,٢ |

• وزن شيكارة الشبة الصلبة هو ٥٠ كجم

تحضير محلول شبة يعادل تركيز ١٠ % صلبة باستخدام الشبة السائلة ذات تركيز ٥٠ %*

| ارتفاع محلول الشبة المطلوب تجهيزه (م) | ارتفاع المياه المضافة (م) | حجم المياه (م٣) | ارتفاع الشبة ٥٠% المضافة (م) | حجم الشبة ٥٠% (م٣) |
|---|---|---|---|---|
| ٤ | ٣,٣٩٤ | ١٣٠,٦٦٩ | ٠,٦٠٦ | ٢٣,٣٣٣ |
| ٣,٩ | ٣,٣١ | ١٢٧,٤٣٥ | ٠,٥٩٠ | ٢٢,٧٥ |
| ٣,٨ | ٣,٢٢٥ | ١٢٤,١٦٢ | ٠,٥٧٥ | ٢٢,١٦٦ |
| ٣,٧ | ٣,١٤ | ١٢٠,٨٩ | ٠,٥٦٠ | ٢١,٥٨٣ |
| ٣,٦ | ٣,٠٥٥ | ١١٧,٦١٧ | ٠,٥٤٥ | ٢٠,٩٩٩ |
| ٣,٥ | ٢,٩٧ | ١١٤,٣٤٥ | ٠,٥٣٠ | ٢٠,٤١٦ |
| ٣,٤ | ٢,٨٨٥ | ١١١,٠٧٢ | ٠,٥١٥ | ١٩,٨٣٣ |
| ٣,٣ | ٢,٨٠ | ١٠٧,٨ | ٠,٥٠٠ | ١٩,٢٤٩ |
| ٣,٢ | ٢,٧١٥ | ١٠٤,٥٢٧ | ٠,٤٨٥ | ١٨,٦٦٤ |
| ٣,١ | ٢,٦٣ | ١٠١,٢٥٥ | ٠,٤٧٠ | ١٨,٠٨٣ |
| ٣ | ٢,٥٤٥ | ٩٧,٩٨٢ | ٠,٤٥٥ | ١٧,٤٩٩ |
| ٢,٩ | ٢,٤٦١ | ٩٤,٧٤٨ | ٠,٤٣٩ | ١٦,٩١٦ |
| ٢,٨ | ٢,٣٧٦ | ٩١,٤٧٦ | ٠,٤٢٤ | ١٦,٣٣٣ |
| ٢,٧ | ٢,٢٩١ | ٨٨,٢٠٣ | ٠,٤٠٩ | ١٥,٧٤٩ |
| ٢,٦ | ٢,٢٠٦ | ٨٤,٩٣١ | ٠,٣٩٤ | ١٥,١٦٦ |
| ٢,٥ | ٢,١٢٢ | ٨١,٦٩٧ | ٠,٣٧٨ | ١٤,٥٨٣ |
| ٢,٤ | ٢,٠٣٦ | ٧٨,٣٨٦ | ٠,٣٦٤ | ١٣,٩٩٩ |
| ٢,٣ | ١,٩٥٢ | ٧٥,١٥٢ | ٠,٣٤٨ | ١٣,٤١٦ |
| ٢,٢ | ١,٨٦٧ | ٧١,٨٧٩ | ٠,٣٣٣ | ١٢,٨٣٣ |
| ٢,١ | ١,٧٨٢ | ٦٨,٦٠٧ | ٠,٣١٨ | ١٢,٢٤٩ |
| ٢ | ١,٦٩٧ | ٦٥,٣٣٤ | ٠,٣٠٣ | ١١,٦٦٦ |
| ١,٩ | ١,٦١٢ | ٦٢,٠٦٢ | ٠,٢٨٨ | ١١,٠٨٣ |

* كثافة الشبة السائلة الخام ١,٣٢ جم / سم٣ في هذا المثال



و يمكن تأكيد تركيز الشبه فى احواض الشبة عن طريق قياس كثافتها و مقارنة القيمة بالجدول الاتى :-

| الكثافة بالهيدروميتر | التركيز المقابل لها |
|---|---|
| ١.٠٣ | %١٢ |
| ١.٠٤ | %١٦ |
| ١.٠٥ | %٢٠ |
| ١.٠٦ | %٢٢ |
| ١.٠٧ | %٢٦ |
| ١.٠٨ | %٣٠ |
| ١.٠٩ | %٣٤ |
| ١.١ | %٣٨ |

## اختبارات التحكم فى التشغيل

مقدمة

- لـ لـ بدء فـ ى إجراءات الـ تحكم فـ ى الـ ثـ سغ يل وتـ حديـ د إضـافـ ة الـ شـ بةو فإنه من الضرورى البدء أولاً فى إجراء تجربة اختبار تحديد الجرعات (Jar test) والتى يتم على ضوء نتائجها تحديد جرعة الشبة المطلوبة إضافتها للمياه .

جهاز تحديد جرعات الشبه

- يـ تكون الـ جهاز، كما هو مـ بـ ين الـ شكل من مجموعه من الـ كؤوس الـ زجاجـ ية و الذى يستخدم لتحديد جرعة الشبه الفعالة معمليا

ويوجد فى كل كاس قلاب صغير، ويتم إدارة هذه القلابات بواسطة عامود إدارة عن طريق تروس ، ويتم إدارة العامود بواسطة محرك كهربائى صغير .

- ويـ تم دوران الـ قلابـ ات عن طريـ ق الـ تروس بـ سرعة تـ ماثـ ل سرعة تـ حرك الماه فى المروقات ويتم التقليب السريع بسرعة ٨٠ ـ ٣٠٠ لفة/ دقيقة لمدة (١٥ ـ ١٢٠ ) ثانيه تخفض الصرعة إلى (٢٠ ـ ٤٠ ) لفة/ دقيقة لمدة (١٥ ـ ٤٥ ) دقيقة بعد إضافة الجرعات المختلفة .

## خطوات تجربة تحديد الجرعات

**١ . تجهيز المحلول**

**فى حالة الشبة السائلة:**

ضع ١٥.٦ ملليلتر من الشبة السائلة (كثافة نوعية ١.٣٢٥) فى قارورة سعة ١٠٠٠ مللى لتر وأكمل القارورة حتى علامة ١٠٠٠ بالماء للحصل على محلول شبة بتركيز ١٠٠٠٠مجم/ لتر ( ١ % )

**ملحوظة:**

تغيير الكثافة النوعية للشبة السائلة حسب الشركة المنتجة.

**فى حالة الشبة الصلبة:**

١. ضع ١٠ جم من الشبة الصلبة سلفات الألومونيوم**(Al2(SO3)3.18H2O)** الى ٦٠٠ مللى لتر مياه فى قارورة سعتها ١٠٠٠ سم٣ وأكمل القارورة حتى علامة ١٠٠٠ سم٣ بالماء. لتحصل على محلول شبة بتركيز ١٠٠٠٠مجم/ لتر (١%)

٢. أجمع ٨ لتر عينة من المياه العكرة .

٣. ضع فى كل كاس من الكئوس الستة كمية مقدارها ١٠٠٠ مللى لتر من المياه العكرة .

٤. ضع الكئوس داخل الجهاز ثم اسقط ريش التقليب فى الكئوس وقلب الماء بسرعة تعادل سرعة المياه بأحواض المحطة .



٥. اضف بحرص وسريعاً بواسطة ماصة جرعة الكلور المقدمة لجميع الكئوس

6. استخدم الماصة فى إضافة جرعات شبه متزايدة بالتوالى لكئوس الاختبار الستة بحرص وبسرعة بقدر الامكان .

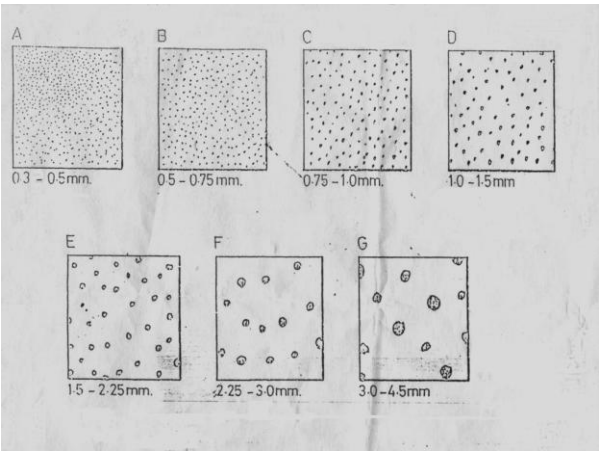
٧. استمر فى التقليب السريع لمدة حوالى دقيقة واحدة من بداية اضافة الشبة .

8. قلل من سرعة التقليب لتصل إلى ٢٠ - ٤٠ لفه/ د ) واستمر فى تقليب المحلول من مدة تتراوح من ٢٠ إلى ٣٠ دقيقة .

لاحظ سرعة بداية تكون الندف فى كل كاس وشكلها وحجمها .

١٠. أوقف عملية التقليب ولاحظ ترسيب الندف بالكئوس لمدة ٥ – ١٠ دقيقة. وعادة تشير العينة ذات الغيوم إلى عدم كفاءة الترويب. أما ذات الترويب الجيدة فتبدو رائقة لأحتوائها على ندف مجمعة ثقيلة الوزن

** سجل النتائج بتصنيفها إلى : ضعيفة / مقبولة / جيدة / ممتازة .




١١. اسحب بواسطة سيفون على شكل حرف **(S)** فى نفس الوقت كمية المياه التى تمثل ارتفاع ٧ سم العلوية من الكاس وهى حوالى نصف لتر تقريباً للكاسات الستة، وجمعها فى عدد ستة مخابير لاجراء باقى الاختبارات عليها كما هو موضح فى الجدول





**نموذج تسجيل نتيجة فحص العينات**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| اسم محطة التنقية: | | | | | | | | |
| اختبار التنديف رقم: | | | | | | | التاريخ: | |
| الكيماويات المساعدة رقم (١): كبريتات الألومنيوم (الشبة) | | | | | | | الزمن: | |
| الكيماويات المساعدة رقم (٢): الكلور | | | | | | | | |
| الكيماويات المساعدة رقم (٣): | | | | | | | | |
| العملية | الوحدة | ١ | ٢ | ٣ | ٤ | ٥ | ٦ | ٧ |
| الشبة بتركيز ١٠ جم/ لتر | | | | | | | | |
| الكلور | ppm | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| جودة التندف: | | | | | | | | |
| الحجم | | | | | | | | |
| الكمية | مل | | | | | | | |
| سرعة الترسيب | | | | | | | | |
| عكارة المياه المروقة | NTU | | | | | | | |
| الطحالب | عدد/ مل | | | | | | | |
| الرقم الهيدروجينى | | | | | | | | |
| الكلور المتبقى | PPM | | | | | | | |
| الروبة %: | | | | | | | | |
| الكمية | | | | | | | | |
| معامل تماسك الروبة | | | | | | | | |
| المياه المرشحة | | | | | | | | |
| العسر | ملجم/ لتر | | | | | | | |
| القلوية | ملجم/ لتر | | | | | | | |
| المواد العضوية | | | | | | | | |
| الألومنيوم المتبقى | | | | | | | | |
| الرقم الهيدروجينى | | | | | | | | |
| العكارة | NTU | | | | | | | |
| أفضل النتائج: | | | | | | | | |
| المادة المساعدة | | PPM | المادة المساعدة | | | | | PPM |
| كبريتات الألومنيوم (الشبة) | | | | | | | | |
| كلور | | | | | | | | |

PPM : جزء فى المليون

## الحسابات اللازمة لتجربة JAR TEST

- تعيين قيمة الG-VALUE للمزج لسريع وتحويلها الى لفة/دقيقة باستخدام الجدول التالى
- اذا تم استخدام المأخذ بعد حقن الكلور لابد من تحديد الفترة الزمنية فى المحطة بين اضافة الكلور والشبة
- يتم تعيين فترة المكث فى المزج السرع
- يتم حساب فترة المكث فى احواض التنديف

فترة المكث= الحجم/التصرف

- يتم حساب قيمة ال G-valueللتنديف
- يتم حساب الحمل السطحى

الحمل السطحى=التصرف/مساحة السطح

**مع مراعاة ان تكون مياة الم؟أخذ متساوية فى الكمية والنوعية فى الكاسات وذلك بالتقليب المستمر للمأخذ بين اضافة كل كاس**

- يتم تحضير محلول الشبة وسحب المياة العكرة فى نفس يوم التجربة
- يتم قياس العناصر المطلوبة لمياة المأخذ
- يتم تحضير الكاسات والادوات التى سيتم سحب العينات بها بعد الترسيب للقياس



**علاقة ال G –Value بعدد اللفات للقلاب السريع**

| م | G-VALUE (S-1) | عدد اللفات rpm |
|---|---|---|
| ١ | 3.3 | 10 |
| ٢ | 9.5 | 20 |
| ٣ | 18 | 30 |
| ٤ | 27 | 40 |
| ٥ | 38 | 50 |
| ٦ | 61 | 70 |
| ٧ | 105.5 | 100 |
| ٨ | 200 | 150 |
| ٩ | 300 | 200 |
| ١٠ | 600 | 300 |
| ١١ | 870 | 400 |
| ١٢ | 1203 | 500 |

## وتوجد عدة عوامل هامة فى تقييم نتائج اختبار تحديد الجرعات :

- معدل تكوين الندف
- شكل جزيئات الندف
- شفافية المياه تحت بين الندف
- حجم هذه الندف
- كمية الندف المتكونة
- معدل ترسيب الندف
- شفافية المياه للطبقة الرائقة

وتختلف تركيزات الشبة المستخدمة فى اختبار الكئوس تبعاً لاختلاف درجة عكارة المياه الخام أو بالتالى كمية الجرعة المطلوبه للمعالجة كما هو موضح بالجدول



## تركيز المواد الكيماوية (الشبة) المستخدمة فى إختبار ( Jar Test)

| الجرعة المطلوبة مللجم / لتر | عدد الجرامات /لتر (التحضير) | ١ مللى من المضاف لكل كأس يساوى | تركيز المحلول % |
|---|---|---|---|
| ١٠ -١ | ١ جم / لتر | ١ مللجم / لتر | % ٠,١ |
| ٥٠-١٠ | ١٠جم / لتر | ١٠ مللجم / لتر | %١ |
| ٥٠٠-٥٠ | ١٠٠جم / لتر | ١٠٠مللجم / لتر | %١٠ |

## اختبارات التحكم فى التشغيل

تـ وجد خمـ سة اخـ تـ بارات تـ ساعد عـ لى الـ تحكم فـ ى عمـ لـ ية الـ ترويـ ب/التندف وهى كالآتى:

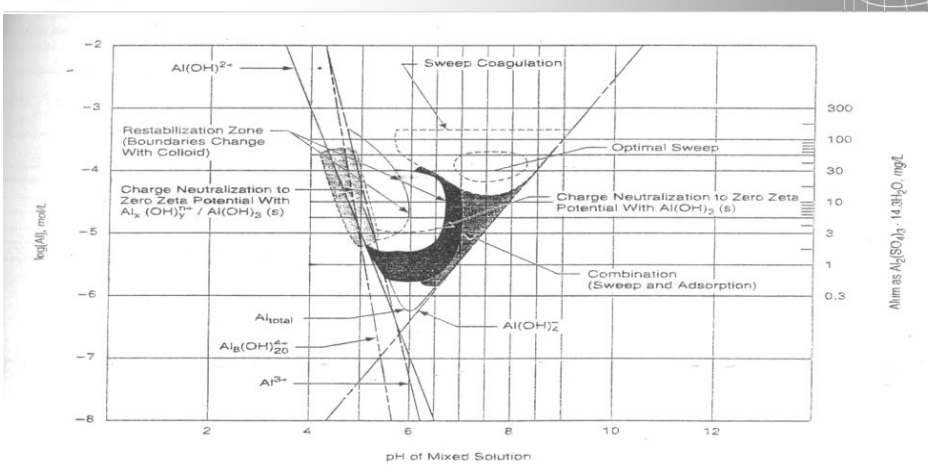
- **اختيار تحديد الجرعات Jar test**
- **اختبار الرقم الهيدروجينى (pH)**
- **قابلية الترشيح**
- **جهد زيتا Zp**
- **الشفافية (أو العكارة)**

## الرقم الهيدروجينى

يـ جب معرفـ ة الـ قائـ م عـ لى الـ تـ شغـ يل لـ درجة الـ تاير ن الـ هـ يدروجـ ينى الـ مـ ثـ لى لـ كل نـ وع من المرويات ويعطى أفضل نتائج تساعده على اختيار المروب الملانم أو يعمل على تصحيحها وضبطها قبل إجراء عملية التر ويب. كما أن قياس الرقم الهيدروجينى (pH)فى عينات المروق تعطى الدلائل الصحيحة على جودة الترويب .








*Source: Amirtharajah and Mills (1982).*

Figure 4-3 Alum coagulation diagram

الشكل السابق هو diagram تم تصميمة لتسهيل فهم عمل الشبة ومن الشكل يتضح الاتى:

- الاس الهيدروجينى والجرعة هما العاملان الرئيسيان المتحكمان فى عمل الشبة مع درجة الحرارة والمواد العضوية والتى تزيد من ذوبان الالمونيوم
- المنطقة خارج حدود المثلث هى منطقة ذوبان للالمونيوم ولا يحدث اى ترويب او تنديف
- المنطقة داخل حدود المثلث هى منطقة عمل الشبة
- مثال :جرعة ١٠٠ جم/م٣ لا تعمل عند pH اعلى من ٩.٥ او اقل من ٤.٥ لتكون الهيدروكسيدات الذائبة بينما تبدأ العمل عند pH اقل من ٩ واعلى من ٤.٥حيث يبدأ تكون الهيدروكسيدات القابلة للترسيب
- بينما جرعة ٣جم/م٣ تبدأ العمل من pH من ٧ – ٥
- كل 1 مجم/لتر شبة صلبة يحتوى على ٠.٠٩ مجم/لتر $AL^{+3}$(تركيز ١٦-١٧%)
- كل 1 مجم/لتر شبة سائلة يحتوى على ٠.٠٤٥ مجم/لت$Al^{+3}$(تركيز ٨%)
- نقطة عدم الذوبان للالمنيوم تحدث عند pH 6-6.2 صيفا وعند pH 6.5-7 شتاءا
- الشبة حمضية وعند اضافتها للماء تؤدى الى انخفاض ال pH



## جهد زيتا (Zp)

يستخدم اختبار جهد زيتا للمساعدة على تعيين أفضل درجة تاين هيدروجينى وأفضل جرعة للمرويات مثل الألومونيوم $Al^{+++}$ لذا يجب مراقبة جهد زيتا (Zp)ليتراوح فى مدى معلوم لانتاج أقل عكارة ويتفاوت هذا المدى من محطة إلى محطة ويتراوح غالباً من 10 – 4 مللى فولت

**درجة الترويب فى نطاقات مختلفة لجهد زيتا**

| متوسط جهد زيتا | درجة الترويب |
|---|---|
| (٣٠) – (صفر) | الحد الأقصى |
| (١-) – (٤-) | ممتازة |
| (٥-) – (١٠-) | متوسطة |
| (١١-) – (٢٠-) | ضعيفة |
| (٢١-) – (٣٠-) | معدومة تقريباً |

## اختبار قابلية الترشيح

- يستخدم اختبار قابلية الترشيح كطريقة لاختبار مدى الفاعلية التى يمكن بها ترشيح مياه مروبة، ويقيس الاختبار مقدار المياه المرشحة فى مدة معينة ويجرى عادة بترشيح كمية صغيرة من مياه المحطة المروبة والمندفة فعلا قبل الترسيب فى مرشح قطره ١٥ سم( مرشح استرشادى) ويحتوى على نفس نوع وعمق وسط الترشيح المستعمل فى مرشحات المحطة ويزود هذا المرشح بجهاز لقياس العكاره ليسجل عكارة المياه الخارجية حيث تجرى قياسات مستمرة لعكارة المياه المرشحة ومن مميزات هذا الاسلوب فى التحكم فى الشغيل ما يلى :
- أن الماء المستعمل فى هذا الاختبار هو الماء المروب فعلا بالمحطة وليس لعينة مماثلة يتم تحضيرها فى المعمل.
- أن الماء عند أية نقطة يستغرق نحو ١٠ دقائق للمرور خلال المرشح ولما كانت مدة المكث فى حوض الترسيب بحجمه الكامل وفى المرشحات الفعلية تستغرق نحو ساعتين إلى ثلاث ساعات لذلك فإن التغيرات فى جرعات المروب يمكن أن تجرى بناء على أداء المرشح قبل حدوث تدهور خطير فيه . وبصفة عامة فإن إجراء



اختبارات (Jar test) واختبار (pH) ودرجة العكارة والشفافية (اختبارات التحكم فى التشغيل ) تعطى دلالة جيدة عن النتائج المتوقعة إلا أن تشغيل المحطة على الواقع قد لا يتطابق مع هذه النتائج ، لذا يجب مراقبة الأداء الفعلى من أجل:

- ملائمة الجرعة المناسبة لنوعية المياه المستخدمة.
- ملائمة الخلط السريع وحسن انشار المادة المروبة.
- ملائمة عملية التنديف البطىء.
- خزانات المياه المروبة والمرشحة.

- ويعتمد التنديف الناجح على الخلط السريع والتام ورغم أن الترويب يحدث فى أقل من ثانية واحدة إلا أن حوض الخلط قد يهيئ مدة مكث تصل إلى ٦٠ ثانية ويجب أن يكون الخلط عن طريق عملية الدوامات التى تحدث فى حركة المياه حتى ينشر المروب فى كل حوض الترويب .
- ويرى البعض أنه خلال العشر ثوان الاولى من نقطة الإضافة يجب أن يخلط المروب اختلاطاً تاماً مع كل قطرة مياه لبدء تكوين ندف فعالة وإلا فإن كفاءة العملية كلها تهبط ويتضح تاثير معظم الاستعمال الخاطئ للمروب عند هذه النقطة. ويمكن تصحيح هذه المشاكل إما بتعديل نقطة الإضافة أو طريقة الخلط .

## ضبط جرعة الشبه

- يتم ضبط جرعة الشبة بقاذفة العكارة فى المياه ويعاد ضبط جرعة الشبة كلما تغيرت العكارة فى المياه بالتنسيق بين مشرف التشغيل وكيميائى المعمل ولا يتم عملية ضبط الجرعة مرة واحدة بل يلزم المتابعة المستمرة لها حسب توجيهات المعمل لتقليل استهلاكها مع المحافظة على أعلى نسة جودة للمياه ، ولو أمكن مثلا تقليل نسبة استهلاك الشبة، فإن ذلك يوفر كثيرأمن تكاليف التشغيل



### طريقة حساب جرعة الشبه

- يفرض أن طلمبة الشبة تعطى ٥١٤ جالون/ ساعة عند مشوار ١٠٠ % المكبس ولنفرض أن المعمل حدد جرعة الشبة ٣٠ جم/م٣ لمروق يعطى ٥٠٠٠ م٣/ ساعة
  - كمية الشبه الصلبة المطلوبة = ٥٠٠٠ X ٣٠ 15000 = جم / ساعة و لكن حيث الشبة المستخدمة سائلة بتركيز ١٠ % ( ١٠ جم شبه صلبة /١٠مل ماء)

∴ كمية الشبة السائلة المطلوبة = ١٥٠٠٠٠ × ١٠٠ / ١٠

= ١٥٠٠٠٠ مل/ ساعة

= ١,٥ م٣ / ساعة

∴ مكبس الطلمبة يعطى ٥١٤ جالون/ ساعة عند مشوار ١٠٠ %

= ٥١٤ × ٣,٨ لتر/ ساعة

= ١,٩٥ م٣ / ساعة

∴ لكى تحصل على جرعة ١,٥ م٣/ ساعة فإنه يلزم ضبط المشوار بنسبة

= ١٠٠ x ١,٥ / ١,٩٥ = ٧٦%

وحيث أن أقصى مشوار للمكبس مقسم إلى ١٠٠ قسم (علامة).

∴ يتم ضبط مشوار للمكبس على العلامة (٧٦).

- ملحوظه: جالون أمريكى = ٣.٨ لتر

## طريقة حساب جرعة الشبه

- بـ فرض أن ط لم بة الـ شـ بة تـ عطى ٥١٤ جالون/ ساعة عند مشوار ١٠٠ % المكبس ولنفرض أن المعمل حدد جرعة الشبة ٣٠ جم/م٣ لمروق يعطى ٥٠٠٠ م٣/ ساعة
- كمية الشبه الصلبة المطلوبة = ٥٠٠٠ X ٣٠ 15000 = جم / ساعة و لكن حيث الشبة المستخدمة سائلة بتركيز ١٠ % ( ١٠ جم شبه صلبة /١٠مل ماء)

∴ كمية الشبة السائلة المطلوبة = ١٥٠٠٠٠ × ١٠٠ / ١٠

= ١٥٠٠٠٠٠ مل/ ساعة

= ١,٥ م٣ / ساعة

∴ مكبس الطلمبة يعطى ٥١٤ جالون/ ساعة عند مشوار ١٠٠ %

= ٥١٤ × ٣,٨ لتر/ ساعة

= ١,٩٥ م٣ / ساعة

∴ لكى تحصل على جرعة ١,٥ م٣/ ساعة فإنه يلزم ضبط المشوار بنسبة

= ١,٥ x ١٠٠ / ١,٩٥ = ٧٦%

وحيث أن أقصى مشوار للمكبس مقسم إلى ١٠٠ قسم (علامة).

∴ يتم ضبط مشوار للمكبس على العلامة (٧٦).

- ملحوظه: جالون أمريكى = ٣.٨ لتر



## خطوات حساب جرعة الطلمبات

- لـ حـ ساب كمـ ية الـ شـ بة الـ تى يـ جب إ ضافـ تها فـ ى الـ ساعة إلـ ى الـ مـ ياه الـ عـ كرة باستخدام طلمبة الحقن فإنه يجب تحديد فتحة تصرف الطلمبة كالآتى:

فتحة تصريف الطلمبة = (كمية المياه العكرة (م٣/س) × الجرعة جم/م٣ × ١٠٠) / (تركيز الشبة (جم/لتر) × سعة الطلمبة (أقصى تصريف) (لتر/س))

وبفرض أن كمية المياه العكرة التى تدخل بيارة التوزيع تساوى ٢٠٠٠ م٣/ساعة وتركيز محلول الشبة المعد للغستخدام ١٠% والجرعة المطلوبة ٢٥جم / م٣، سعة طلمبة الشبة ١٢٠٠ لتر/ س.

∴ تكون كمية الشبة المطلوبة فى الساعة = $\frac{٢٥ \times ٢٠٠٠}{١٠}$ x ١٠٠

= ٥٠٠,٠٠٠ مل = ٥٠٠ لتر

وبالتالى فإنه يتم ضبط الطلمبة على فتحة مقدارها = $\frac{١٠٠ \times ٥٠٠}{١٢٠٠}$

= ٤١,٦% تقريباً

أى أن الطلمبة تضبط على ٤١,٦% من سعتها لتعطى ٥٠٠ لتر/ ساعة بجرعة قدرها ٢٥ جم/ م٣ لكل ٢٠٠٠ م٣ مياه.



$Al_2 (SO_4)3 + 3Ca (HCO_3)2$ ------- $2Al (OH)_3 + 3CaSO_4 + 6CO_2$

$Al_2 (SO_4)_3 + 3 NaCO_3 + 3H_2O$------- $2Al_2 (OH)_3 + 3Na_2SO_4 + 3CO_2$

$Al_2 (SO4)_3 + 3Ca (OH)_2$ ---------- $2Al (OH)_3 + 3CaSO_4$

من المعادلة السابقة يتضح ان

كل ١ مجم/لتر شبة صلبة يقلل القلوية بمقدار ٠.٤٥ مجم/لتر as CaCO3

كل ١ مجم/لتر شبة سائلة يقلل القلوية بمقدار ٠.٢٢٥ مجم/لتر as CaCO3

يتم قياس فرق القلوية بين العكرة والمزج السريع او بداية حوض التنديف

## □ المواصفات القاسيه المصرية
## ٤ - ١٩٨٢
## كبريتات الالمونيوم ( الشبه الصلبة) المستعملة فى عمليات معالجة مياه الشرب

**مقدمة**

هذه المواصفات تلغى وتحل محل المواصفات القياسية المصرية رقم ٤ لسنة ١٩٥٨ التى سبق قيدها ونشرها بالسجل الرسمى فى ٣/١٠/١٨٥٨.

١. **المجال**

تحدد هذه المواصفات القياسية خواص كبريتات الألومونيوم (الشبه) المستعملة فى تنقية مياه الشرب التى تنطبق عليها الصيغة الجزئية ( $Al_2(SO_4)_3.18H_2O$ )وتحضر بالتعامل الكيمائى بين حمض الكبريتيك مع أحد الخامات الغنية بالألومنيوم مثل البوكسيت (أكسيد الألومنيوم )

٢. **الوصف**

يجب أن تكون كبريتات الألومنيوم (الشبه) المستعملة فى تنقية مياه الشرب.

- من النوع النقى
- تكون على هيئة قوالب أو قطع صغيرة أو حبيبات
- تكون خالية من المواد الضارة (تحدد ها المعامل المركزية بوزارة الصحة)





## □ ٣ . الخواص الطبيعية و الكيميائية :-

- لاتزيد نسبة المواد غير القابلة للذوبان فى الماء على ٠.٥
- لاتقل نسبة أكسيد الألومنيوم عن ١٥.٣ %
- لا تزيد نسة الحديديك ٠.٩ %
- لا يزيد الزرنيخ على ٥٠ جزء فى المليون

## اخذ العينات

- تقسيم العينة المرسلة بطريقة التربيع ثم يؤخذ منها حوالى ١٠٠ جرام وتطحن ثم تجرى عليها الفحوص اللازمة .

- **اولا:**

أ. اذا كانت كمية الرسالة لا تزيد على ٢٠٠٠٠ عبوة فيؤخذ منها عدداً من العبوات طبق للجدول الأتى:

| حجم الرسالة بالعدد | | حجم العينة بالعدد |
|---|---|---|
| من | إلى | |
| ٢ | ٨ | ٢ |
| ٩ | ٢٧ | ٣ |
| ٢٨ | ٦٤ | ٤ |
| ٦٥ | ١٢٥ | ٥ |
| ١٢٦ | ٢١٦ | ٦ |
| ٢١٧ | ٣٤٣ | ٧ |
| ٣٤٤ | ٥١٢ | ٨ |
| ٥١٣ | ٦٢٩ | ٩ |
| ٧٣٠ | ١٠٠٠ | ١٠ |
| ١٠٠٠ | ١٠٠٠٠ | ١١ |
| ١٠٠٠٠ | ٢٠٠٠٠ | ١٢ |



ويراعى عند أخذ العبوات أن تكون من مواضع مختلفة من الكمية .

- ب. اذا كانت الكمية أكثر من ٢٠٠٠٠ عبوة تقسم الكمية إلى مجموعات كل مجموعة ١٠٠٠ عبوة ويؤخذ من كل منها عشر عبوات (١%) وتجمع العبوات المختارة ويؤخذ منها ٥ % من عددها حسبما اتفق فتكون هذه العبوات هى التى تمثل الكمية .

## ثانيا:-

- يؤخذ من كل عبوة حوالى نصف كيلو جرام وذلك بعمل ثقب أو فتحة فى العبوة ويراعى اختيار المواضع بحيث تكون غير متشابهة أو متماثلة فى العبوات المختلفة ثم تجمع العينات جميعها وتخلط مع بعضها خلط جيدا ثم تجزأ بطريقة التربيع حتى نحصل على العينة ممثلة وزنها حوالى ثلاثة كيلو جرامات

## ثالثا:-

تقسم العينة الممثلة إلى ثلاث عينات تزن الواحدة منها حوالى واحد كيلو جرام وتوضع كل منها داخل وعاء جاف نظيف من البلاستك أو الزجاج ثم تقفل جميعها قفلا جيدا أو تختم بالشمع الاحمر ويكتب عليها بخط واضح التاريخ الذى أخذت فيه العينة رقم الرسالة وتوقيع المكلف بأخذها وترسل أحداها للمعمل الكيميائى لاجراء التحليل اللازمة عليها ويحتفظ بالثانية لدى المشترى والثالثة لدى المورد وذلك لاحتمال طلب اعادة التحليل من أحد طرفى التعاقد.





## ٥. طرق الاختبار

**طريقة تعين المواد غير القابلة للزوبان فى الماء**

يوزن بدقة حوالى ٢٠ جم من العينة وتوضع فى كأس من الزجاج وتذاب فى حوالى ٢٠٠مللي لتر من الماء المقطر الساخن وتقلب لبضع دقائق ثم ترشح خلال بوتقة من الزجاج الملبد رقم ٤ أو بوتقة الالندوم موزونة وزنا دقيقا بعد تجفيفها عند درجة حرارة ١٠٠-١٠٥ مْ إلى أن يثبت الوزن وتحسب النسبة المئوية للمواد غير القابلة للذوبان فى الماء من العلاقة الاتية

$$\text{النسبة المئوية} = \text{و} \times \frac{١٠٠}{\text{و}}$$

**حيث:**

و١ = وزن المتبقى فى البوتقة

و = وزن العينة بالجرام

**طريقة تعين أكسيد الالمونيوم و الحديد:**

ينقل ناتج الترشيح من العملية السابقة بعد أن يبرد لحرارة المعمل إلى قنينة قياس سعتها ١ لتر ثم تكمل بالماء المقطر إلى العلا مة وبواسطة ماصة ٢٥ ملليلتر لتوضع فى كأس من الزجاج سعة ٤٠٠ ملليلتر وتخفف بالماء المقطر ليصير حجمها حوالى ٢٠٠ ملليلتر ثم يضاف اليه حوالى ٥ جم من كلوريد النوشادر.

- ويـ ـ ق لب الـ محـ لول جـ يدا حـ تى الإذاب ـة وبـ عد ذلـ ك يـ ضاف ١ ملليلتر من حمض الهيدركلوريك المركز وأربع نقط من حمض النيتريك المركز ويقلب المحلول ببط ء لبضع دقائق وذلك لأكسدة الحديد ثم يضاف نقطتان من دليل أحمر الميثيل .

يضاف محلول هيدروكسيد النشادر المخفف بنسبة ١:١ تدريجيا مع التقليب المستمر إلى أن يبدأ الراسب فى الظهور ويستمر فى إضافة النشادر نقطة فنقطة حتى يظهر اللون الأصفر. تغلى محتويات الكأس لمدة دقيقة واحدة يتحول خلالها اللون الأصفر إلى اللون الأحمر الوردى ويرشح المحلول فورا باستعمال ورق ترشيح من النوع سريع الرشح مثل (ورق واتمان ٤١) مع مداومة غسل الراسب على ورق الترشيح بمحلول ٢ فى المائة من كلوريد النشادر الساخن إلى أن يتخلص تماما من الكبريتات من ناتج الترشيح .





ويجب مراعاة عدم زيادة مدة الغلى عن اللازم أو التأخير فى بدء عملية الترشيح. تجفف ورقة الترشيح بما عليها من راسب بعد ذلك فى فرن هوائى درجة حرارته ١٠٠ مْ ثم تنقل إلى بوتقة من البلاتين موزونة وزنا دقيقا وتحرق با ستعمال اللهب العادى اولا ثم تنقل بعد تمام حرق ورقة الترشيح إلى فرن درجة حرارته حوالى ١٠٠٠ مْ أو با ستعمال البورى لمدة ساعة ثم تنقل البوتقة إلى المجفف وتنزل فيه حتى متبرد .

- تـ وزن وتـ عاد إلـ ى الـ فرن والـ مجـ ـفف معـ تـ كرار الـ عمـ لـ ية إلـ ى أن يثبت الوزن:-
- تـ حـ سب الـ نـ سة الـ مـ ئوي ـة لـ هذيـ ن الا كـ سـ يديـ ن من الـ علاقـ ة الاتـ ـية :

$$\text{النسبة المئوية} = \text{و} \times \frac{١٠٠}{\text{و}} \times \frac{١٠٠}{٢٥}$$

و١ = وزن الراسب المتكون

و = وزن العينة بالجرامات

## طريقة تعيين أكسيد الحديد

يحضر محلول قياس يحتوى على ٠.١ جم من أكسيد الحديديك من اللتر وذلك بأذابة ٠.٤٩ جم من كبريتات الحديدوز النشادرى فى ٥٠ ملليلتر من الماء المقطر و ٢٠ ملليلتر من حمص الكبريتك المركز ويسخن المحلول تسخينا هينا مع إضافة برمنجنات البوتاسيوم ( ٠.١ ع نقطة فنقطة إلى أن يتم أكسدة الحديد ويعرف ذلك عند ظهور لون وردى خفيف لا يزول بعد دقيقة واحدة ثم ينقل المحلول إلى قنينة قياس سعتها لتر. ويضاف من الماء المقطر الكمية المناسبة إلى العلامة ويصير كل١ ملليلتر من هذا المحلول محتويا على ٠.١مليجرام من أكسيد الحديدك ( ٠.٠٠٠١ جرام أكسيد الحديدك ) وباستعمال طريقة التقدير المقارن بالألوان المعروفة يمكن تقدير كمية الحديد وذلك بأخذ ١ ملليلتر من ناتج الترشيح المذكور فى صدر العملية السابقة وتوضع فى انبوبة نسلر وتخفف إلى ٥ ملليلتر بالماء ويضاف اليها ٢.٥ ملليلتر من حمض الهيدروكلوريك المخفف بنسبة ١:٣ ثم نضع نقط من محلول برمنجنات البوتاسيوم حتى يصير اللون ورديا لمدة ٥ دقائق ثم يكمل بالماء المقطر إلى ٤٥ ملليلتر.



ثم يضاف ٥ ملليلتر من محلول ثيوسيانات البوتاسيوم وفى أنابيب نسلر أخرى مشابهة لها تماما فى السعة والقطر يوضع ٠.٢ ، ٠.٤ ، ٠.٦ ، ٠.٨ ، ١.٠ ، ملليلتر .. الخ من محلول أكسيد الحديديك القياسى السابق شرحة فى صدر هذا البند ويضاف اليها نفس نسب الكواشف وتكمل جميعها إلى ٥٠ ملليلتر بالماء المقطر وترج محتويات الأنابيب رجأ جيدأ وتنقل الأنبوبة التى بها العينة إلى المكان المخصص لها فى جهاز قياس الالوان ثم تنقل إلى الجزء الاخر المخصص للمحاول القياسى الانابيب المذكورة فيما قبل لانتخاب الواحدة منها التى يتماثل لونها مع لونه الانبوبة التى بها العينة .

وتحسب النسبة المئوية لاكسيد الحديديك فى العينة من العلاقة الاتية:

$$\text{النسبة المئوية} = \text{ح} \times \frac{١}{١٠٠٠٠} \times \frac{١٠٠٠٠}{\text{و}} \times ١٠٠$$

**حيث:**

ح = حجم محلول الحديد القياسى بالملليلترات فى الأنبوبة المنتخبة

و = وزن العينة بالجرامات

وبطرح هذه النتيجة من النسبة المئوية لاكسيدى الالومنيوم والحديد تنتج النسبة المئوية لاكسيد الالومنيوم على حدة .



**المواصفات القياسية المصرية**

**١٧٠٠-١٩٨٩**

**الشبه السائلة المستخدمة فى عمليات معالجة مياه الشرب**

**١- المجال**

تختص هذه المواصفات القياسية بمواصفات الشبة السائلة المستخدمة فى عمليات معالجة مياه الشرب ، وتحضر بالتعامل الكيميائي بين حمض الكبريتيك مع أحد الخامات الغنية بالألومنيوم مثل الكاولين أو غيره .

**٢. الوصف**

تكون الشبة السائلة المستخدمة فى عمليات معالجة مياه الشرب:

- من النوع النقى.
- خالية من المواد الضارة بالصحة (تحددها المعامل المركزية بوزارة الصحة )
- شفافة بحيث تسمح بقراءة أجهزة القياس بسهولة.

## ٣. الخواص الطبيعة والكيميا ئية

| الخواص | النسبة المئوية |
|---|---|
| – المواد غير القابلة للذوبان فى الماء | لا تزيد على ٠,٢ % |
| – أكسيد الألومنيوم | لا تقل عن ٨ % |
| – الحديد | لا تزيد على ٠,٣٥ % |
| – الزرنيخ | لا تزيد على ٠,٠٠٥ % |
| – الأس الهيدروجينى | لا يقل عن ٣ |
| – المعادن الثقيلة محسوبة كرصاص | لا تزيد على ٠,٠١ % |
| – النيتروجين محسوب كنيتروجين | لا تزيد على ٠,٠١% |
| – الوزن النوعى عند 20°م | ١,٣% |



## عينات الاختبار

**سحب العينات**

- تسحب عينات عشوائية متجانسة وممثلة بمعرفة الجهات المختصة من كل خزان من خزانات التخزين الموجودة لدى المنتج والمستهلك على السواء وكذا من صهارج شاحنات النقل وكذلك عند الا ستخدام لدى المستهلك.

**حجم العينات**

- تسحب العينات بحجم لايقل عن ٢ لتر وتمزج جيدا ويعبأ منها ثلاثة عبوات بحجم لا يقل عن١/٢ لتر فى وعاء زجاجى محكم الغلق غير منفذ للهواء ثم تحرز وتختم بالشمع الاحمر .
- يكتب على كل وعاء بخط واضح التاريخ الذى أخذت فيه العينه ورقم الرسالة وتوقيع المشترى أو المورد أو مندوبيهما .
- ترسل عبوة للمعمل الكيميانى المختص لاجراء التحليل اللازمة عليها .
- تحفظ العبوتان الباقيتان واحدة طرف المورد والأخرى طرف المشترى .

**النقل والتخزين**

- تنقل الشبة السائلة فى صهاريج قياسية مناسبة إلى مكان الا ستخدام وتخزن فى خزانات مناسبة لا تتأثر بالشب ولاتؤثر فيها .

# الطرق التقليدية فى معالجة مياه الشرب





## تطهير مياه الشرب

## مقدمة

- يعتبر الماء ثانى أهم العناصر الضرورية للانسان بعد الأكسجين الذي نستنشقه من الهواء. وكما أن الماء لازم لاستمرار الحياة فقد يكون سببا في القضاء عليها إذا استعمل ملوثا بجراثيم الأمراض منل التيفود والدوسنتاريا والكوليرا أو غيرها .
- والاهتمام بالماء وما يرتبط به من أمراض ليس وليد العصر الحديث فلقد أوصى به بقراط ( اله الطب عند القدماء) بغلي الماء الذى يستعمل للشرب كما ظهر واضحا على اللوحات الأثرية التى خلفا المصريين القدماء أنهم كانوا يقومون بغلى الماء قبل استخدامه للشرب .



ولقد أصبح من الصعب الحصول على مصدر المياه الآمن نظرا لأن بعض المدن تستعمل المجارى المائية المجاورة لها سواء كانت - انهارا أو بحارا أو مصارف - أو بحيرات لصرف مخلفاتها السائلة عليها سواء قبل أو بعد علاجها جزئيا أو كليا ، لذلك فان من واجب الحكومات أن تراقب مل هذه الحالات وعليها ايجاد الحلول اللازمة لتحفظ للمجرى المائي سلامته من أي تلوث قد يضر بالصحة العامة أو يؤذى الثروة المائية .

- يتضح مما سبق إن تطهير مياه الشرب أمر حيوي وضروري حرصا علي صحة المواطنين وضمانا لسلامتهم. وعملية التطهير في معالجة المياه تهدف في المقام الأول إلى التخلص من كل أنواع الكائنات الحية الدقيقة .

## عملية تطهير المياه

- عملية تطهير المياه هى أهم خطوة من خطوات معالجة المياه فهى التى تضفى عليه صفة الصلاحية وتؤمنه ضد الأمراض المعدية المنقولة عن طريقه وتتم عملية تطهير المياه بطرق متعددة منها:-

**المعالجة الحرارية**

- وهي أول طريقة تم اكتشافها منذ سنوات عديدة ويتم فيها غليان الماء، باستخدام الغليات، و لا تستخدم هذه الطريقة في إنتاج كميات كبيرة من المياه نظرا لارتفاع تكلفة الطاقة المطلوبة لها وإن كانت تستخدم في تحلية المياه المالحة .

**المعالجة الاشعاعية**

- وتتم عن طريق مرور المياه على لمبة تنتج أشعة فوق بنفسجية ويجب أن تكون المياه خالية من العكارة وقريبة جدا من اللمبه المشعة لذا لا تستخدم هذه الطريقة إلا فى إنتاج الكميات الصغيرة من المياه

**المعالجة الكيماوية**

- يمكن استخدام العديد من المواد الكيمائية فى تطهير المياه ومنها: الكلور (لكلورة )، ثانى أكسيد الكلورو برمنجانات البوتاسيوم ، الأوزون الجير، البروم، اليود.



## الشروط الواجب توافرها فى المواد المطهرة :

١. أن تكون قاتلة للجراثيم ولا تؤثر على صحة الإنسان.

٢. أن تكون رخيصة الثمن ومتوفرة محليا أو يسهل استيرادها طوال العام

٣. أن تكون استعمالها سهل ومأمونا.

٤. أن تكون تخزينها سهل وآمناً.

## التطهير بالكلور

- عملية إضافة الكلور من أكثر الطرق انتشارا في عمليات تطهير المياه وقد كان لظهور عملية الكلوره في أواخر القرن الماضي اثر كبير في القضاء بدرجة كبيرة علي الأمراض التي ينقلها الماء وذلك بأقل التكاليف وابسط المعدات واقل عدد من العاملين ولولا عملية الكلورة لانتشرت أوبئة الكوليرا والتيفود والتى كان لظهور الكلور واستخدامه الفضل الأكبر فى القضاء عليهما .
- ويتميز التطهير بواسطة الكلور بسهولة استعماله، وكذلك سهولة الحكم على مدي فاعليته بالتأكد من بقاء قدرا من الكلور في الماء بعد فترة من إضافته .



**ولا يعرف علي وجه التحديد كيف يعمل الكلورين علي قتل البكتريا في الماء وإن كان المعتقد أن مركبات الكلور التي تتكون عندما يضاف الكلور ين إلى الماء تقضى على بعض الأنزيمات الضرورية للعمليات الحيوية فى هذه الكائنات و لقد تاكد عدم صحة الاعتقاد السائد بان البكتريا تتاكسد نتيجة اضافة الكلورين وذلك لان عملية التأكسد تحتاج الى كميات هائلة من الكلور، في حين أن التطهير الناتج الآن لا يتطلب إلا كميات ضئيلة منه. وهذا لا يمنع أن الكلو رين عابل مؤكسد قوي وعندما يستخدم كميات كافية فإنه يوقف نمو الطحالب فى المرشحات .**

- **والكلور عنصر كيماوي ينتج خلال صناعة الصودا الكاوية ويدخل الكلور في كثير من الصناعات المدنية والحربية منها علي سبيل المثال صناعة البترول والمعادن والورق والمنسوجات والصناعات الكيميائية والصناعات الغذائية كما يستعمل الكلور في معالجة مياه الصرف الصحى وتطهير مياه الشرب .**

## صناعة الكلور

- يتم تحضير الكلور في الصناعة بواسطة التحليل الكهربي لمحلول كلوريد الصوديوم (ملح الطعام). ويتجمع الغاز عند القطب الموجب ويتم سحبة حيث تتم إسالته ويحفظ في خزانات كبيرة إلى أن تتم عملية تعبئته في الاسطوانات المستخدمه في محطات المياه ومصانع النسيج و خلافه .

## خصائص الكلور

- يمكن أن يتواجد الكلور مثله مثل أى مادة فى ثلاث صور مختلفة: السائلة، الغازية. الصلبة ولكل منها خصائصه .



## غاز الكلور

- غاز لونه أصفر مائل إلى الخضرة وهو أثقل من الهواء مرتين و نصف له رائحة مميزة شديدة النفاذية ووزنه الذرى ٣٥.٥
- الكلور الغازي ضعيف القابلية للذوبان في الماء (٦.٧ جم/لتر ماء عند ٢٠مْ ، ١٤.٨ جم/لتر ماء عند صفر مْ ولذلك لا ينبغي رش الماء على الكلور المتسرب ، كما أن حفظ محلول الكلور في درجة حرارة باردة يحافظ على . تركيزه لفترة طويلة .
- وبرغم أن الكلور ليس مادة متهبة قابلة للاشتعال أو الانفجار، إلا انه يمكن أن يساعد على الاشتعال تحت ظروف معينة .
  كذلك فإن الكلور الجاف ليس مادة مسببة للتأكل ، إلا انه يتحول إلى ذلك بشكل مؤثر إذا ما تعرض للرطوبة.
- غاز الكلور سام ومهيج للاغشية المخاطية المبطنة للانف والعين والجلد والرئتين ويسبب سعال وصعوبة فى التنفس.

يسال بالتبريد عند -٣٤.١ مْ، ضغط جوى ا بار( أو يسال تحت ضغط عالي )٧كجم/ سم ٢ ، في درجات الحرارة العادية( ولذلك يحفظ وينقل علي هيئة غاز مسال بالضغط في اسطوانات من الصلب تختلف سعتها من خمسين الى ١٠٠٠ كيلوجوام وتتوقف العبوة المستعملة علي الكمية المستهلكة في محطة المياه.

في حالة وجود تسرب لغاز الكلور فى الجو فإن ٣ جزء فى المليون هى اقل نسبة يمكن حسها بالشم ء وعند ١٥ جزء في المليون يصبح تأثيرها مهيج للعين والرئة، ويصبح خطرا إذا ما استشق لفترة من ٣٠ - ٦٠ دقيقه عند تركيز من٤٠ - ٦٠ جزء فى المليون، وإذا زاد التركيز فى الجو ليصل إلى ١٠٠٠ جزء في المليون فإنه يصبح مميت. أما الكمية المسموح بها لغاز الكلور في الجو وتكون آمنة لفترة الوردية الواحدة (ثماني ساعات) لا تتجاوز تركيز ٠ . ١ جزء في المليون.



## الكلور السائل

- **عبارة عن محلول ذو قى كهرماني اللون وهو أثقل من الماء مرة ونصف تقريبا. وللكلور السائل معامل تمدد عالي، إذ يزداد حجمه بسرعة كبيرة بازدياد درجة الحرارة، حيث يزداد تمدد السائل ليملاء الأسطوانة بالكامل عندما ترتفع درجة الحرارة إلى ٦٧.٥ مْ ولذلك يلتزم دائما بعدم ملء اسطوانات الكلور بأكثر من ٨٥ % من حجمها. وحيث أن الكلور السائل يتبخر بسرعة شديدة إذا ما تعرض للهواء الجوى، لذلك فهو نادرا ما يرى في صورته السائلة.**
- **وعند تبخر الكلور السائل فإن وحدة الحجوم الواحدة منه تنتج حوالي ٤٥٦ وحدة حجم من الغاز النقى عند درجة حرارة ١٥ مْ وضغط ٧١٠ مم زئبق**
- **و بالتالى عند حدوث تسرب فى اسطوانة الكلور يتحتم تعديل وضع الاسطوانة بحيث تكون منطقة التسرب فى اعلاه لكى يتسرب غاز الكلور و ليس السائل**

**الكلور الصلب**

- نظرا لان الكور السائل يتجمد عند درجة حرارة منخفضة جدا ( -١٠٢ م°) فهو نادرا ما يوجد في صورته الصلبة ، غير انه يتواجد متحدا مع بعض العناصر الأخرى فى صورة مركبات على هيئة بودرة أو حبيبات .

- **مركبات الكلور**
- بخلاف الكلور النقى يتواجد الكلور علي هيئة مركبات سائلة أو صلبة :

**محلول الكلور**

ويسمي كيماويا (هيبوكلوريت الصوديوم وهو محلول يحتوي علي حوالي ١٥% من الكلور الحر ويباع في المحلات العامة كمنظف تحت أسماء مختلفة، ويمكن استعماله كمطهر للمياه بشرط أن لا يضاف إليه أي أنواع من المنظفات ويضاف مباشرة على الماء سواء بالصب المباشر من وعاء أو باستعمال مضخة مناسبة. (لا يستعمل بكثرة بالإضافة إلى آن محلوله يسبب تآكك في المواسير)





- **مسحوق أو أقراص الكلور:**

**ويسمي كيماوياً (هيبوكلوريت الكالسيوم) و هذه المادة تحتوى علي حوالي ٦٠ – ٧٠ % من الكلور الحر.**

- **ويمتاز عن المسحوق المبيض بارتفاع نسبة الكلور الفعال وبان نسبة الكلور الفعال لا تتأثر بالتخزين لفترات ت ليست بالطويلة. وعند الاستعمال يحضر محلول مركز منه ثم يضاف إلى الماء بالجرعات اللازمه بواسطة أجهزة خاصة .**

- **المسحوق المبيض:**

**ويسمي أحيانا (كلوريد الجير أو الجير المكلور وهو مسحوق ابيض مائل للاصفرار، له رائحة قوية نفاذة، الجديد منه يحتوي علي ٣٢ % من وزنة كلور فعال. إلا أن هذه النسبة تأخذ فى النقصان بمضي الوقت خصوصا إذا تعرض للجو أو للضوء، ولذلك يجب حفظة في عبوات خاصة محكمة القفل ، كما يجب اختباره لمعرفة نسبة الكلور الفعال قبل الاستخدام حتى يمكن تقدير الكمية التي تعطي جرعة الكلور المطلوبة.**

- **وعموما فإن استعمال مركبات الكلور الصلبة غير شائع في عمليات التطهير لمياه نظرا لمتاعب التشغيل إلا انه يستعمل فى الحالات الآتية:-**

**١ . تطهير شبكات مواسير توزيع المياه بعد إنشائها او إصلاحها.**

**٢ . تطهير مرشحات وخزانات المياه**

**٣. في حالات الطوارئ مثل حالات الفيضانات.**

## استخدامات الكلور فى محطات المياه

يستخدم الكلور فى محطات المياه لغرض التطهير و للقضاء على مشاكل الطعم و الرائحة و لاكسدة الشوائب الكيميائية

**١ . التطهير:**

يستخدم الكلور فى محطات مياه الشرب كمادة مطهرة. إذ أن إضافة اكلور تتسبب فى قتل البكتريا الضارة والمسببة للامراض. ولذلك تحدد اشتراطات جهات الصحة المختلفة ضرورة إضافة جرعات وافية من الكلور للماء تكفل توافر الكلور المتبقى حت ى نهايات شبكة التوزيع ( فى حدود ٢ . ٠ ملجم/لتر)

**٢ . القضاء على مشاكل الطعم والرائحة:**

قد يستخدم الكلور للقضاء على بعض مشكلات الطعم والرائحة فى مياه الشرب. إذ أن معظم مشاكل الطعم والرائحة تتسبب عن بعض النباتات المجهرية الدقيقة التى تتواجد فى مصادر المياه السطحية. فيتفاعل الكلور كمادة مبيده للطحالب ويقضى على تلك النباتات المسببة للطعم والرائحة. إلا أن هناك أنواع قليلة من الطحالب لا تتفاعل مع الكلور بالشكل المرغوب ويترتب على ذلك زيادة حدة الروائح الطعم غير المقبول للماء.

**٣ . الأكسدة:**

يقوم الكلور بأكسدة عدد من الشوائب الكيميائية الموجودة فى الماء كالحديد.

المنجنيز ، النيترات ، الأمونيا وكبريتيد الهدروجين . وبالتالى ازالتها من الماء.

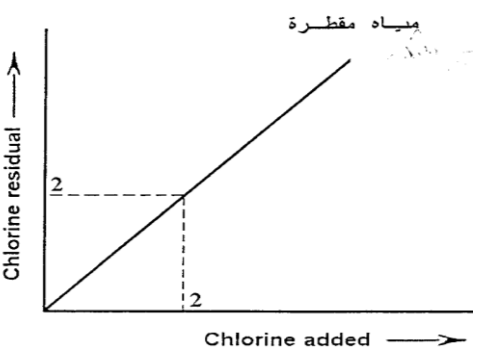
وتتواجد تلك الشوائب عادة فى مصادر المياه الجوفية .





## نظرية عمل الكلور

لفهم تفاعلات الكلور في المياه الطبيعية . نفرض مبدئياً أن التفاعل سيحدث في مياه مقطرة . فإن كمية الكلور الحر المتبقية تتعلق مباشرة بكمية الكلور المضافة( الجرعة .)



وعلى سبيل المثال: إذا اضيف جرعة كلور بمقدار ٢ مجم/لتر لتلك المياه ستعطى نفس القيمة ( ٢ مجم/لتر) ككلور متبقى حر بعد انتهاء فترة المكث

**وسيتم التفاعل كالتالى:**

كلور + ماء ← حامض الهيبوكلورز + حامض الهيدروكلوريك

$CL_2 + H_2O \rightarrow HOCl + HCl$ ..... (1)

**نواتج التفاعل مركبات ضعيفة لا تلبث ان تتفكك إلى نواتج أخرى كالتالي:**

حامض الهيبوكلورز ← أيون الهيبوكلوريت + ايون هيدروجين

(حامض ضعيف يتفكك)

$HOCl \rightarrow OCl^- + H^+$ ..... (2)

وحامض الهيدروكلوريك ← ايون كلور + ايون هيدروجين

$HCl \rightarrow Cl^- + H^+$ ..... (3)

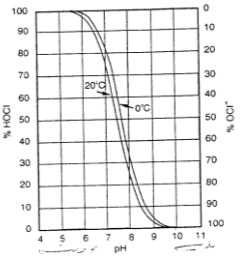


ويمثل حمض الهيبوكلورز HOCl أحد شكلين من أشكال الكلور الحر المتبقى وهو الأكثر فاعلية في عملية التطهر وعند تحلله كما في المعادله ( ٢ ) السابقه ينتج ايون الهيبوكلورت-OCl وهو الشكل الثانى من أشكال الكلور الحر التبقي وتأثيره في عمله التطهير تعادل ١% فقط من تأثير حمض الهيبوكلورز.

| الشكل | فاعلية الكلور مقارنة بحمض الهيبوكلورز |
|---|---|
| HOCl | 1 |
| OCl – | 1/100 |
| NCl3 | - |
| NHCl2 | 1/80 |
| NH2CL | 1/150 |

## تأثير pH على التفاعل:

يتأثر التفاعل بالأس الهيدروجيني تأثيرا كبيرا فكلما كان الوسط اقرب للحامضية زاد حامض الهيبوكلوروز وكلما كان الوسط اقرب للقاعدية زادت أيونات الهيبوكلوريت.




**فمثلا :-**

**عند الاس الهيدروجينى ٦.٧ تكون النسبة**

- ٢٠ % ملح هيبوكلوريت +٠ ٨ % حمض هيبوكلوروز

**عند الاس الهيدروجينى ٧.٣ تكون النسبة**

- ٤٠ % ملح هيبوكلوريت +٠ ٦ % حمض هيبوكلوروز

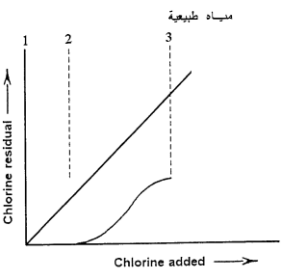
**عند الاس الهيدروجينى ٨.٣ تكون النسبة**

- ٨٠ % ملح هيبوكلوريت + ٢٠ % حمض هيبوكلوروز





يتبين مما سبق انه قد يوجد كلور بالعينة. ولكن فاعليته تختلف بالزيادة او النقصان تبعا لاختلاف درجة تركيز الأس الهيدروجينى




أما بالنسبة للمياه الطبيعية فكما نعلم أن بها مواد ذائبة وشوائب عالقة وبالتالى فان تفاعل الكلور مع تلك المواد والشوائب سيتداخل ويؤثر على كمية الكلور المتبقى الحر كنتيجة لذلك.

- وعلى سبيل المثال: إذا احتوى الماء على حديد، منجنيز، أمونيا، نيتريت ، أو أي مادة عضوية فسوف يتفاعل الكلور المضاف معها ليعطى انحراف في المنحنى يناسب تلك المتفاعلات ودرجة تركيزها بالمياه

من الشكل:- المسافة بين النقطتين ٢ و ١ الكلور المضاف يتحد مباشرة مع الحديد و المنجنيز و النيتريت كمواد مختزلة ولا يظهر تبعا لذلك أي كلور متبقى حتى الانتهاء من أكسدة كل تلك المواد بواسطه الكلور.

باستمرار إضافة الكلور نلاحظ انه على امتداد المسافة بين النقصتن ٣ ، ٢ يبدأ الكلور في التفاعل مع الأمونيا والمادة العضويه ليشكل مركب الكلورامين والمركبات الكلو وعضوية وهذه النواتج من أشكال الكلور تعرف بالكلور المتبقى المتحد ولكن فاعلة هذه الأ شكال كعامل مطهر تكون اضعف من الكلور الحر.

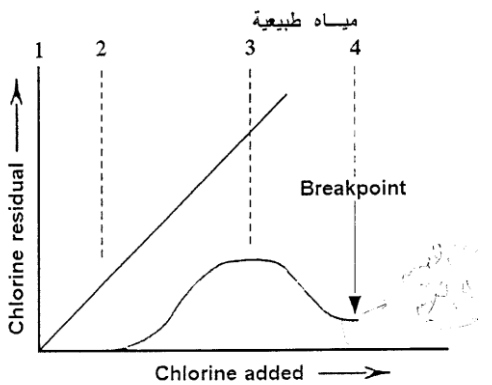
وفيما بين النقطتين ٣ و ٢ غالبا يكون أحادى الكلورامين هو شكل الكلور المتبقى كناتج التفاعل:

أمونيا + حمض الهيبوكلورز ← أحادى الكلورامين + ماء

$NH_3 + HOCl \rightarrow NH_2Cl + H_2O \quad (pH > 7) \ldots\ldots (4)$



يوضح الشكل التالى انه بمتابعة إضافة كلور للمياه ، يبدأ الكلور المتبقى من النقطة ٣ حتى النقطة ٤ في الانخفاض وذلك كنيجة لبدأ أكسدة مركبات أحادى الكلورامين الى مركبات ثاني وثالث الكلورامين و حتى تمام الأكسدة وتكون غاز النيتروجين .



ويوضح ذلك التفاعلات التالية:

في حالة إذا ما كان $NH_3$ بكمية وفيرة:

$$NH_2Cl + HOCl \rightarrow NHCl_2 + H_2O \quad (4 < pH < 6)..... (5)$$

$$NHCl_2 + HOCl \rightarrow NCl_3 + H_2O \quad (pH < 3)..... \quad (6)$$

ونلاحظ من التفاعلات السابقة انه عند pH اكبر من ٧ يتكون أحادى الكلورامين ، وعندpH 6 يبدأ ثاني الكلورامين ببطئ في التكوين ويظهر بوضوح بين ٦ >pH>٧ أما ثالث الكلورامين فيتكون عند pH أقل من ٣ . كما أن الكلورامين ممكن أن يتك,ن مع أي Amine Organic منل البروتين، حيث يظهر الكلورامين بسرعة أولا ثم يعقبه ببطء ظهور ثاني وثالث الكلورامين مع الاحتياج إلى فترة تلامس أطول حتى يتم التفاعل.

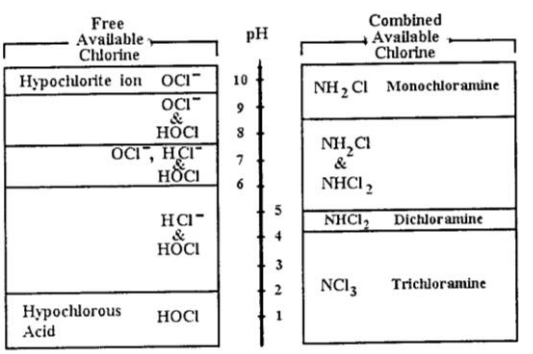
إذا كان كمية $NH_3$ في الماء كافية فإن التفاعل سيكون ثابت ، وإذا ما كان كمية الكلور المضافة اكبر مع $NH_3$ فان الكلورامين سيتكسر إلى النير وجين كما في المعادلة التالية : -





$$2NH_2Cl + HOCl \rightarrow N_2 + 3\,HCl + H_2O \quad ..... (7)$$

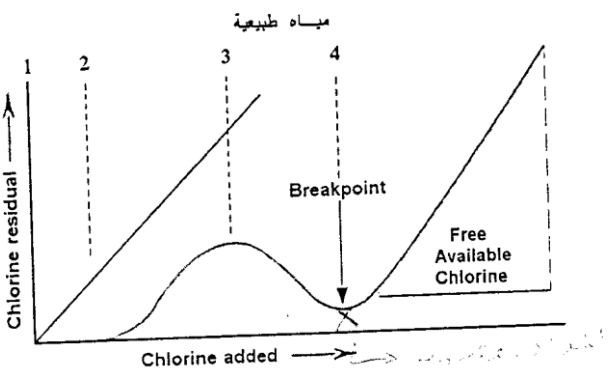
٦٠) يوضح علاقة pH بأشكال الكلـــور المحتمـــل تواجـــدها بالميـــاه



ويختلف مفعول كل نوع من مركبات الكلور السابقة عن الآخر في عمليات التطهير حيث يلاحظ أن الكلور الحر اقوي بكثير في قتل البكتريا عن الكلور المتحد. ويلا حظ أيضا أنه يلزم استخدام مقدار من الكلور المتحد اكبر ٢٥ مرة من الكلور الحر لتحقيق نفس القدرة علي القتل في نفس المدة الزمنية

- علاوة علي ذلك فان التطهير بالكلورامين عملية بطيئة وتحتاج إلى فترة تلامس اطول.
- الـ نـ قطة ٤ هى أدنى قيمة يصل لها الكلور وتعرف باسم نقطة الانكسار. ثم يبدأ بعدها ظهور كلور حر متبقي ويزيد بزيادة جرعة الكلور المضافة . بمعنى انه بعد نقطة الانكسار أي إضافة للكلور تظهر في صورة كلور حر متبقي، وقيمة الكلور الحر المتبقى يجب أن تمثل ٨٥ ـ ٩٠ % من مجموع الكلور المتبقى (حر + متحد) بالمياه المعالجة .






يجب أن نتذكر أن الكلور المتبقى المتحد يمثل مركبات ثنائى و ثلاثى الكلورامين والمركبات الكورو- عضوية )As THMs

## تتوقف فاعلية الكلور فى قتل البكتريا على العوامل الآتية:-

**١ . جرعة الكلور:**

بديهي إن فاعلية الكلور فى القضاء علي البكتريا تزيد بازدياد جرعة الكلور المضافة الى المياه.

**٢ . مدة التفاعل بين الكلور والماء ( فترة التلامس)**

تزيد فاعلية الكلور كلما طال من التلامس . و نظرا لان مقاومة البكتريا للكلور باختلاف أنواعها . فوجد انه يجب أن تمر ثلاثين دقيقة على الأقل من إضافة الكلور وحتى استعمال الماء كفترة تلامس. وقد وجد أن قدرة الكلور على قتل البكتريا بالماء تتأثر تأثراً كبيراً بالعاملين السابقين (الجرعة، ووقت التلامس ) طبقا للعلاقة التالية:

Kill α CXT

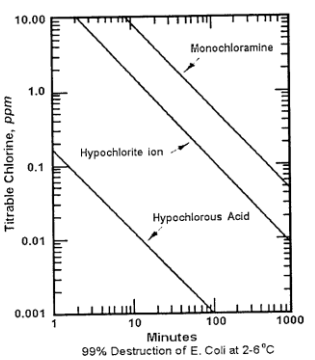
ويؤثر كل منهما على الآخر فبزيادة تركيز الكلور يقل الوقت اللازم للقتل والعكس بالعكس.مع الأخذ في الاعتبار أن الكلور المتحد تأثيره اقل من الحر في عملية التطهير ولذلك يلزمه فترة تلامس أطول لإتمام عملية قتل البكتريا الموجودة بالماء.

من المهم معرفة فترة التلامس الملائمة لنوع الكلور المتبقى الموجود بالماء ، فمثلا نعتمد على الكلور الحر المتبقى في التطهير عندما يكون الوقت المقطوع من نقطة إضافة الكلور حتى نقطة استهلاكه قصير. الشكل ( يبين العلاقة بين الوقت اللازم لقتل بكتريا E Coli بنسبة ٩٩ % عند درجة حرارة ٢ - ٦ م٠ ونوع الكلور المتبقى بالماء .

**تذكر انه : يجب أن يتوفر ٢ . ٠ مجم/لتر كلور حر فى أطراف الشبكة التى تتغذى من محطة المعالجة.**








**٣ درجة الحراره:**

بالرغم من أن درجة حرارة المياه المنخفضة تحتفظ أكثر بالكلور إلا أن الحرارة العالية أكثر فاعلية، ولذلك يجب تغيير جرعة الكلور مع تغير الفصول للحفاظ على نسبة الكلور المتبقى بالمياه، بمعنى إن جرعة الكلور تزيد بانخفاض درجة الحرارة للحصول علي نفس كفاءة التطهير.

**٤. درجة تركيز أيون الهيدروجين:**

لقد تبين أن المياه ذات التركيز الهيدروجيني المنخفض تلزهما جرعات كلور اصغر من المياه ذات التركيزالهيدروجين المرتفع للحصول علي نفس كفاءة التطهير.وتفسير ذلك ان الأس الهيدروجيني هو الذي يحدد النسبة بين تركيزي كل من $HOCl - OCl^-$ ويوضح الشكل السابق تلك العلاقة. لاحظ من الشكل أيضاً أن درجة الحرارة تأثيرها ضعيف على نسبة التحلل بالمقارنة بتأثير الأس الهيدروجيني.

□ا سد تخدام مرك بات الـه يـ بوك لوريـ ت فـ ي عم لـ يات الـ تطه ير ت رف ع من قيمة pH قليلا أما استخدام غاز الكلور فيخفض من pH ، ولذلك من الأهمية متابعة القياس والتحكم في درجة الأس الهيدروجيني للمياه للمحافظة على درجة تأثير الكلور في عملية التطهير .

**٥. عكارة الماء:**

كلما زادت عكارة الماء زادت جرعة الكلور اللازمة إذ أن الميكروبات قد تحتمي بالمواد المسببة للعكارة من تأثير الكلور.





## ٦. قلوية وحامضية الماء:

وتقل فاعلية الكلور بزيادة قلوية الماء ولذلك يلزم استخدام جرعات كلور عالية كلما ارتفعت قلوية الماء.

## ٧ □. وجود المركبات الآزوتية في الماء:

عند تواجد مركبات الأمونيا في المياه تقل فاعلية الكلور في قتل الكائنات الحية في الماء ولذا يلزم إضافة جرعات اكبر أو إطالة وقت التفاعل بين الكلور والماء ( فترة التلامس )

## ٨. وجود مركبات الحديد والمنجنيز:

وهذه أيضا تحد من فاعلية الكلور في قتل البكتريا.

**اضافة الكلور**

- يتم إضافة الكلور للمياه بإحدى طريقتين:
  - إضا فة الكلور بجرعات عالية ثم إزالة الكلور الراند.
  - إضافة الكلور بعد تحديد النسبة بواسطة التجارب المعملية الدقيقة.
- **أولاً: إضافة الكلور بجرعات عالية ثم إزالة الكلور الزائد :-**

والمقصود بذلك إضافة الكلور بجرعات زائدة عن حد الطلب، وبهذا نضمن الحصول على كفاءة وفاعلية عالية لعملية التطهير وممكن أن نضطر إلى ذلك فى حاله الطوارئ أو إذا ما كان هناك شك في حدوث تلوث بكتيري لمصدر المياه.

- **و تتميز هذه الطريقة بالاتى :-**
  - كفاءة وفاعلية عالية لتأثير الكلور علي البكتريا.
  - أكسدة الكلور للمواد العضوية التي قد تتواجد في الماء.
  - الحد من الطعم والرائحة التي قد توجد في الماء.
  - إبادة الكائنات الحية الدقيقة التي تقاوم الجرعات العادية للكلور، علي انه يلزم إزالة الكلور الزائد بعد التأكد من تمام قتل الكلور للبكتريا.





**وطرق إزالة الكلور الزائد :**

١. إضافة ثاني أكسيد الكبريت إلى الماء بجرعات حوالي ١.٥ جزء فى المليون لكل جزء في المليون من الكلور المراد إزالته وفي هذه الحالة يتفاعل ثاني أكسيد الكبريت مع الكلور الزائد كما هو موضح بالمعادلة:

$$SO_2 + H_2O + Cl_2 \rightarrow 2HCl + H_2SO_4$$

كميات حامض الكبريتيك وحامض الهيدروكوريك الناتجة من التفاعل ضئيلة جدا ولا أهمية لها كما يجب أن تمر حوالي ١٥ دقيقة بعد إضافة ثانى اكسيد الكبريت قبل استعمال المياه.

٢. إضافة ثيوسلفات الصوديوم إلى الماء ليتفاعل مع الكلور الزائد كما هو موضح بالمعادلة :

$$2\,Na_2S_2O_3 + Cl_2 \rightarrow Na_2S_4O_6 + 2\,NaCl$$

٣. إضافة كبريتيت الصوديوم إلى الماء ليتفاعل مع الكلور الزائد كما هو موضح بالمعادلة:

$$Na_2SO_3 + Cl_2 + H_2O \rightarrow Na_2SO_4 + 2\,HCl$$

٤. تخزين الماء في أحواض مكشونة لمدة ثلاث أو أربع ساعات قبل الا ستعمال وفي هذه الفترة يتصاعد الكلور الزائد فى الجو .

٥. ترشيح الماء فى طبقة من الكربون المنشط الذى يمتص الكلور الزائد

٦. مزج المياه المضاف إليها جرعات عالية من الكلور بمياه لم يضاف إليها الكلور فتتعادلان .





**ثانياً: إضافة الكلور بعد تحديد النسبة بواسطة التجارب المعملية**

تحديد احتياج الكلور باختبار Break point

احتياج الكلور هو مقياس لمقدار الكلور الواجب إضافته لوحدة الحجوم من المياه تحت شروط محدودة للاس الهيدروجينى ودرجة الحرارة وفترة التلامس حتي يتم التفاعل الكامل بجميع مواد الكلور القابلة للتفاعل فى الماء ويحقق كلور حر متبقي، ويحسب احتياج الكلور بأنه الفرق بين كمية الكلور المستخدم والكلور المتبقى في المياه عند فترة التلامس

## المصطلحات الفنية الخاصة بالكلور

### جرعة الكلور

- تعرف جرعة الكلور بأنها اقل كمية كلور تضاف إلى وحدة حجم من الماء تكفى للقضاء على اللكائنات الحية، وينتج عنها كلور متبتقى في حدود معينة (٠.٢ مجم/لتر)
  ووحدة الجرعة ملجم كلور/لترماء أو جم كلور/ م٣ماء.
- و يتم تحديد الجرعة المثلى للكلور عن طريق تجارب معملية حسب نوعية المياه المراد تنقيتها

### كمية الكلور المطلوب اضافتها للماء

- هى حاصل ضرب جرعة الكلورX كميه المياه وهى غالباً تحدد فى الساعة ووحدتها كجم/ساعة





$$\text{كمية الكلور المطلوبة} = \frac{٢٠٠٠٠\ م^٣/س \times ٥\ جم/م^٣}{١٠٠٠} = ١٠٠\ كجم/س$$

وفى حالة إضافة هيبوكلوريت الكالسيوم تركيز ٦٥% بدلاً من الكلور دون حدوث أى تأثير على المياه، فإنه يلزم أولاً معرفة كمية الكلور المطلوب إضافتها.

كمية الهيبوكلوريت التى تعوض كمية الكلور =

$$\frac{\text{كمية الكلور} \times ١٠٠}{\text{نسبة تركيز الهيبوكلوريت}}$$

**مثـــال:**

ما هى كمية هيبوكلوريت الكالسيوم (٦٥%) التى تضاف إلى المياه بدلاً من ١٠٠ كجم كلور.

$$\text{كمية هيبوكلوريت الكالسيوم (٦٥\%)} = \frac{١٠٠ \times ١٠٠}{٦٥} = ١٥٣,٨٥ \text{ كجم}$$



**الكلور المستهلك**

يعرف الكلور المستهلك بأنه الفرق بين كمية الكلور المضاف للماء وكمية الكلور الحر أو المتحد المتبقى في الماء فى فترة تلامس محددة

الكلور المستهلك = كمية الكلور المضاف – كمية الكلور المتبقى

**مـ ثال:**

ما هى كمية الكلور المستهلك إذا كان الكلور المتبقى ٢ . ٠ جم/ م$^٣$ بعد إضافة ١٠٠ كجم كلور لكمية مياه مقداره ٢٠٠٠٠ م$^٣$ ؟

**الحل**

الكلور المستهلك = كمية الكلور المضاف – كمية الكلور المتبقى

كمـ ية الـ كـ لور الـ مـ تـ بـ قى =٢ . ٠ جم/ م$^٣$ X ٢٠٠٠٠ م$^٣$ = ٤٠٠٠ جم = ٤ كجم

اذا الكلور المستهلك = ١٠٠ – ٤ = ٩٦ كجم

هذه الكمية استهلكت بواسطة الكائنات الحية والمواد العضوية وغير العضوية والشوائب.. إلخ.

ولابد من وجود كلور متبقى بعد كل هذه الاستهلاكات. وهذا دليل على أنه لم يعد هناك أى كائنات حية أو خلافه وأن الماء اصبح خاليا منها و معقم تماما

## الكلور المتبقى

- **هناك نوعان من الكلور المتبقى**

**١ . الكلور المتبقى المتحد:**

ينتج عن إضافة قدر من الكلور يكفى فقط للاتحاد مع الامونيا الموجودة بالماء . وبرغم أن تلك البقايا المتحدة تحمل قدرات أكسده تفوق قدرة الكلور الحر إلا أن فعاليتها كمادة مطهرة تقل عن فاعلية الكلور الحر. وللحصول على الكلور المتبقى المتحد لا يضاف إلى الماء إلا قدر الكلور الذى يكفى

- لـ لاتـ حاد مع الأمونـ يا الـ موجودة فـ ى الـ ماء ، فإذا كان الماء يحتوى على قدر ضئيل من الأمونيا لا يكفى للاتحاد مع الكلور فإنه يتم إضافة الأمونيا أيضاً إلى الماء .





**٢ . الكلورالمتبقى الحر:**

ينتج عن إضافة الكلور إلى الماء بالقدر الذى يكفى لاستهلاك الأمونيا الموجودة فى الماء والقضاء عليها. والكلور المتبقى الحر كما ذكرنا قبل هذا أكثر فعالية من الكلور المتبقى المتحد كمادة مطهرة.

**اماكن إضافة الكلور في محطات التنقية**

- يمكن إضافة الكلور إلى الماء في أكثر من موقع في محطات التنقية تبعا لحالة كل محطة وكذلك تبعا لصفات الماء المعالج في كل حالة وتبعا لتجارب وخبرات المشرف علي التشغيل.

اضافة الكلور المبدئى

- اى حقن الكلور بعد عملية الحصول على المياه من المصد ر مباشرة وقبل الدخول الى عملية التنقية (المروقات والمرشحات )
- **تتميز هذه الطريقة بالاتى :-**
  - خفض تعداد البكتريا في الماء قبل وصولها إلى المرشح مما يخفف الحمل البكتيرى على المرشح.
  - تطهير رمل المرشح نظرا لمرور المياه بما فيها من كلور فى مسام الرمل أثناء عمله الترشح.
  - كفاءه عاليه في إزاله اللون من الماء.
  - نقص في كميات الكيماويات المروبه إذا أضيف الكلور قبل أحواض الترسيب.
  - كفاءة عايله في إزاله الطعم والرائحه من الماء.
  - الحد من نمو الكائنات الحيه الدقيقة داخل المرشح.





**اضافة الكلور النهائى**

- أي إضافة الكلور إلى الماء بعد مرحلة الترشيح أي عند مدخل خزان المياه النقية ، وهي طريقة سهلة في تشغيلها ويكون الكلور أكثر فاعلية علي البكتريا بسبب خلو الماء من أي عكارة أو شوائب.

يضاف جرعة زائدة من الكلور النهائي في الخزانات الملحقة بالمحطات وذلك لضمان خروج الكلور بنسبة معينة مطلوبة إذا كانت شبكات مواسير التوزيع تمتد الى مسافات بعيدة ويخشى من تواجد البكتريا فى الأطراف البعيدة منها وحتى يصل الكلور ولو بنسبة ضئيلة إلى آخر متر في الشبكة.

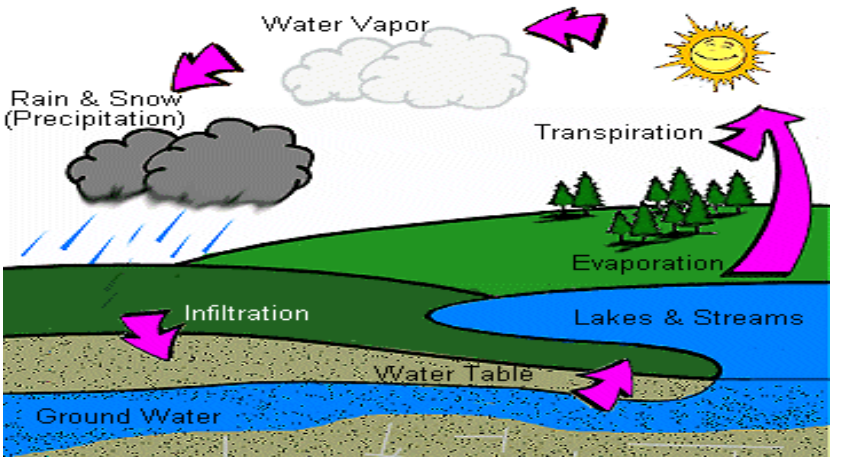
## إضافة الكلور في اكثر من موقع

- وهذه الطريقة تتبع إذا كانت المياه رائقة والتلوث البكتيري عالي نسبيا إذ يحسن في هذه الحالة إضافة الكلور في أكثر من نقطة علي مسار الماء في محطة التنقية لضمان كفاءة عملية الكلوره كما تستعمل هذه الطريقة إذا خزنت المياه المرشحة في خزانات مكشو فةففى مثل هذه الحالة يجب إضافة الكلور في مخارج المياه من الخزانات .
- ** **تذكر انه من نتائج أضافه المواد المؤكسدة (الكلور) للمياه :**
  ١. تكسير أغلب المركبات المسببة للون .
  ٢. أكسدة الحديد والمنجنيز ومعظم المركبات الكبريتية
  ٣. تكسير بعض المركبات المسببه للرائحه .
  ٤. تكسير بعض المر كبات الحلقية .
  ٥. القضاء علي عدد كبير من البكتريا .



# تقنيات معالجة مياه الشرب
# مصادر وخصائص المياه

# الدورة الهيدرولوجية للمياه



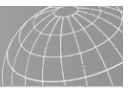



# مصادر المياه

١-الأمطار ٢-البحار ٣-المحيطات
٤-البحيرات ٥-الأنهار ٦-المياه الجوفية

**خصائص المياه**

١. خصائص طبيعية
٢. خصائص كيماوية
٣. خصائص بيولوجية



# الخصائص الطبيعية

١. درجة الحرارة
٢. العكارة
٣. اللون
٤. الطعم
٥. الرائحة

# الخصائص الكيميائية

١. الأس الهيدروجيني
٢. العسر الكلي
٣. الأكسجين الذائب
٤. المواد الذائبة
٥. القلوية
٦. الحمضية
٧. المواد العضوية بأنواعها

# الخصائص البيولوجية

وهي عبارة عن ما تحويه المياه من بكتريا وفيروسات وطحالب وطفيليات ضارة بصحة الإنسان ، ويؤدي اكتشاف هذه البكتريا والفيروسات إلى وضع النظام السليم لتعقيم المياه بما يكفل قتل هذه الكائنات الدقيقة المسببة للأمراض





# الاشتراطات الصحية في المياه

## المواصفات الواجب توافرها في مياه الشرب والاستخدام المنزلي

### الخواص الطبيعية

| م | الخاصية | الحد الأقصى المسموح به |
|---|---|---|
| ١ | اللون | معدوم |
| ٢ | الطعم | مقبول |
| ٣ | الرائحة | معدومة |
| ٤ | العكارة | ١ وحدة عكارة |
| ٥ | الرقم الهيدروجيني PH | ٦.٥-٨.٥ |

## مواد غير عضوية لها تأثير علي الاستساغة و الاستخدامات المنزلية .

| م | المادة | الحد الأقصى المسموح به مليجرام /لتر |
|---|---|---|
| ١ | الأملاح الذائبة عند ١٢٠ ْ م | ١٠٠٠ |
| ٢ | عسر كلي $caco_3$ | ٥٠٠ |
| ٣ | عسر كالسيوم $caco_3$ | ٣٥٠ |
| ٤ | عسر ماغنسيوم $caco_3$ as | ١٥٠ |
| ٥ | كبريتات $so_4$ | ٢٥٠ |
| ٦ | كلوريدات cl- | ٢٥٠ |
| ٧ | حديد Fe | ٠.٣ |
| ٨ | منجنيز mn | ٠.٤ |
| ٩ | نحاس Cu | ٢ |
| ١٠ | زنك Zn | ٣ |
| ١١ | صوديوم Na | ٢.١ |
| ١٢ | الومنيوم Al | ٠.٢ |



## مواد كيميائية لها تأثير علي الصحة العامة

| م | المادة | الحد الأقصى المسموح به مليجرام /لتر |
|---|---|---|
| ١ | الرصاص pb | ٠.٠١ |
| ٢ | الزئبق Hg | ٠.٠٠١ |
| ٣ | الزرنيخ As | ٠.٠١ |
| ٤ | السيانيد Cn | ٠.٠٥ |
| ٥ | الكادميوم Cd | ٠.٠٠٣ |
| ٦ | سيلينيوم SE | ٠.٠١ |
| ٧ | الكروميوم Cr | ٠.٠٥ |
| ٨ | الأمونيا $NH_3$ | ٠.٥ |
| ٩ | النترات $NO_3$ | ٤٥ |
| ١٠ | النيتريت NO2 | ٠.٢ |
| ١١ | الفلوريدات F | ٠.٨ |
| ١٢ | الانتيمون Sb | ٠.٠٢ |
| ١٣ | الباريوم Ba | ٠.٧ |
| ١٤ | البورون B | ٠.٥ |
| ١٥ | النيكل NI | ٠.٠٢ |
| ١٦ | المولبدنوم Mo | ٠.٠٧ |

# أنواع المواد الدخيلة على المياه

- **مواد ذائبة ( Dissolved Matters )**
  **وأهمها أملاح كربونات و بيكربونات وكبريتات و كلوريدات الكالسيوم و الماغنسيوم والصوديوم ، وكذا أملاح مركبات الحديد و المنجنيز و السيلكا وعلاوة على الغازات الذائبة .**
- **مواد عالقة (Suspended Matters )**
  **الطين والرمل والمواد النباتية والحيوانية الميكروسكوبية وأنواع البكتريا**
- **مواد غروية (Colloidal Matters )**





# التأثيرات غير المرغوبة لبعض المواد الدخيلة على المياه

**المواد الذاتية**

- **أملاح الكالسيوم و الماغنسيوم ( العسر )**
- **أملاح الصوديوم (القلوية والطعم )**
- **الغازات الذائبة (ثاني أكسيد الكربون - كبريتيد الأيدروجين – الأكسجين )**

## مواد عالقة

•البكتريا : بعضها يسبب أمراضاً

•الطحالب: تسبب لوناً ، وطعماً ،ورائحة

•الطمي: يسبب عكارة

مواد غروية

•أكسيد الحديد : يسبب لوناً أحمر

•المنجنيز: يسبب لوناً أسود أو بني



## المواد العضوية

تسبب لوناً وطعما ووجود بعض هذه المواد في المياه يجعلها غير نقية أو غير صالحة للاستعمال وقد تسبب الأمراض.

**والعاملين في مجال إنتاج مياه الشرب يجب عليهم**
**إنتاج مياه تلبي متطلبات الجودة مع مراعاة التشغيل**
**الاقتصادي للمرفق**





# أنظمة معالجة مياه الشرب

# معالجة المياه الجوفية

**تختلف أنظمة معالجة المياه الجوفية والمياه السطحية تبغاً لاختلاف الملوثات المراد إزالتها، يتم معالجة المياه الجوفية إذا كانت تحتوي على مكونات تحول دون استخدامها مباشرة مثل أملاح الحديد و المنجنيز أو أملاح تسبب العسر مثل أملاح الكربونات و البيكربونات أو أملاح الكبريتات**

## يمكن تقسيم المياه الجوفية من حيث الجودة الي ثلاث مجموعات

١. مياه جوفية هوائية
٢. مياه جوفية لا هوائية نسبيا
٣. مياه جوفية لا هوائية



# المياه الجوفية الهوائية

**Aggressive water**
**Hard water**

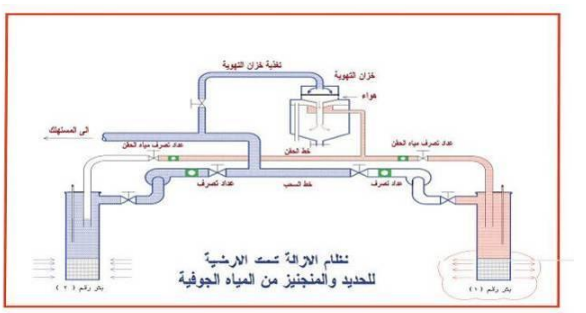
*Figure 4 - Aeration above limestone filter*

# المياه الجوفية اللا هوائيه نسبياً

**مشكلات الحديد و المنجنيز**

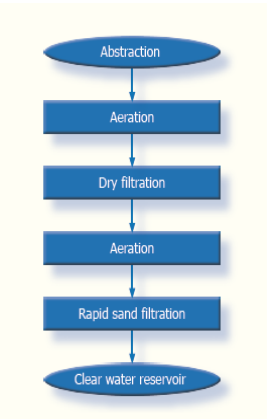
*Figure 7 - Treatment of slightly anaerobic groundwa-ter*

# التهوية خلال التربة وقبل استخراج المياه
# ( BERMAN )



# المياه جوفية اللا هوائية

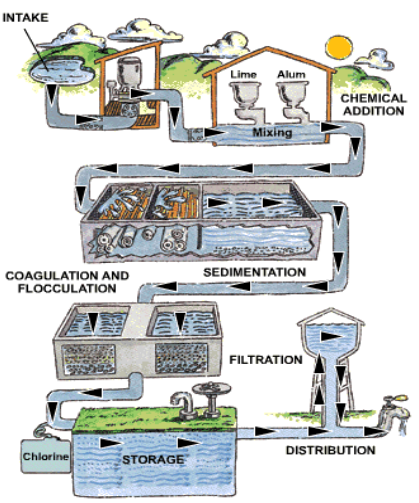
**مشكلات الحديد و المنجنيز و الامونيا**








# معالجة المياه السطحية




**المعالجة التقليدية**
**( المزايا و العيوب )**

# مزايا و عيوب محطات المعالجة التقليدية

**العيوب**

١. المساحة
٢. تكلفة الإنشاء
٣. تكلفة التشغيل
٤. الحاجة غلى عمالة مدربة

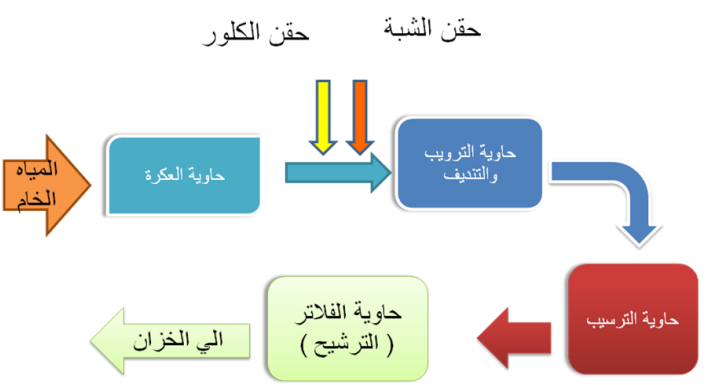
**المزايا**

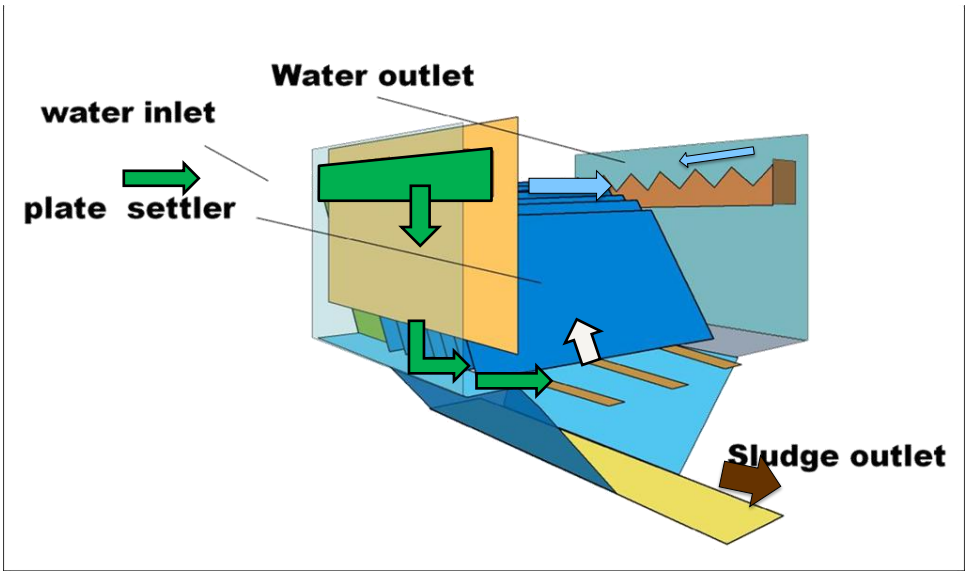
١. جودة المياه المنتجة
٢. المرونة
٣. الكفاءة في مواجهة تغيرات مياه المأخذ
٤. إمكانية التطوير





## محطات الترشيح السريع










# محطات الترشيح السريع
# المزايا و العيوب

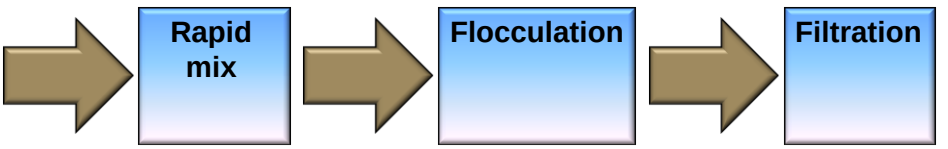
**المزايا**

- قلة التكلفة
- سهولة الإنشاء
- إمكان نقلها من مكان لآخر
- المرونة

**العيوب**

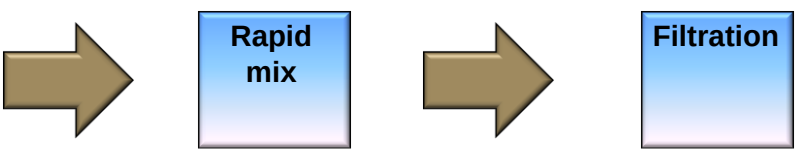
- أعمال الصيانة
- التأثر بتغيرات المياه الخام
- الحاجة الي المراقبة الدائمة
- الحاجة إلى العمالة المدربة

## الترشيح المباشر *direct filtration*






## الترشيح بالتلامس contact filtration



## الترشيح المباشر و الترشيح بالتلامس
## المزايا والعيوب

**المزايا**

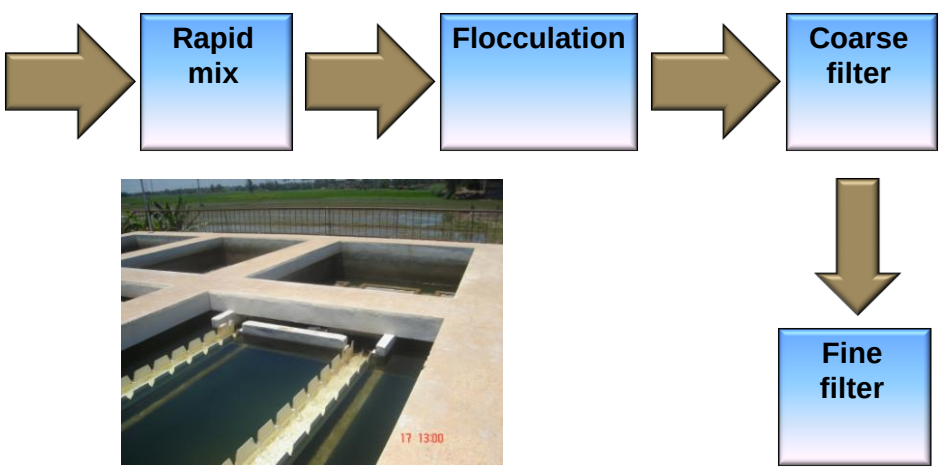
- قلة تكلفة الإنشاء
- قلة تكلفة التشغيل
- صغر المساحة المطلوبة
- مناسب للمياه الخام ذات العكارات المنخفضة

**العيوب**

- لا يناسب المياه ذات العكارات العالية
- يتاثر بتغير المياه الخام
- ضرورة المراقبة المستمرة
- يجب مراقبة المياه الخام من حيث العدد الطحلبي



## الترشيح ذو المرحلتين two stage filtration



## الترشيح ذو المرحلتين two stage filtration
## المزايا والعيوب

**المزايا**

- قلة تكلفة الإنشاء
- قلة تكلفة التشغيل
- صغر المساحة المطلوبة
- يمكنه مواجهة المياه الخام ذات العكارات المرتفعة

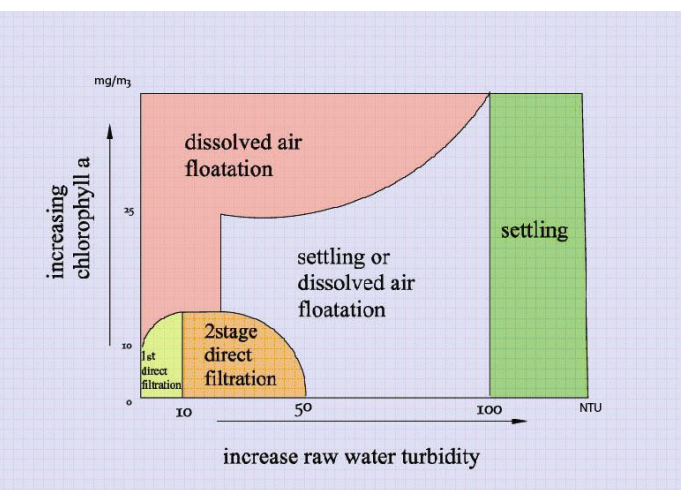
**العيوب**

- يتأثر بتغير المياه الخام
- ضرورة المراقبة المستمرة
- يجب مراقبة المياه الخام من حيث العدد الطحلبي





## اختيار نظام المعالجة



# تقنيات معالجة مياه الشرب

## عمليات المعالجة

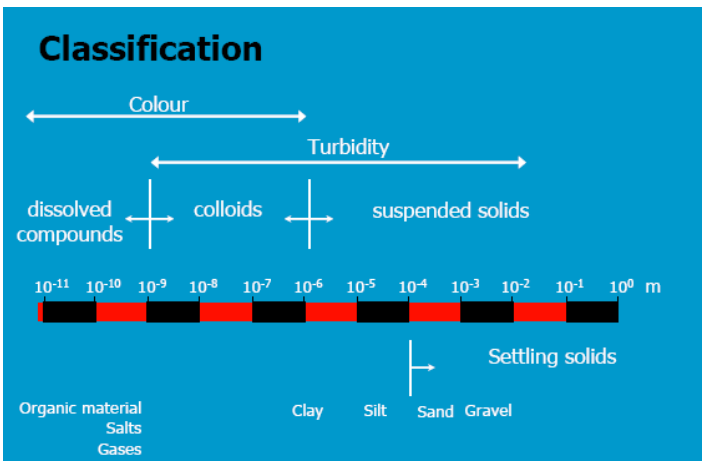




### أولاً :فصل الأجسام الطافية والعالقة بالمأخذ

# ثانيا:عمليتي الترويب و التنديف

**طبيعة الشوائب**








# سرعة الترسيب

| Diameter (m) | Types of particles | Settling time over 30 cm |
|---|---|---|
| $1.10^{-2}$ | gravel | 0.3 sec |
| $1.10^{-3}$ | coarse sand | 3 sec |
| $1.10^{-4}$ | fine sand | 38 sec |
| $1.10^{-5}$ | silt | 33 min |
| $1.10^{-6}$ | bacteria | 35 hours |
| $1.10^{-7}$ | clay | 230 days |
| $1.10^{-8}$ | colloids | 63 years |

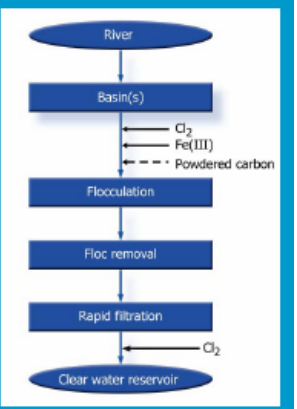
# Principle of coagulation & flocculation

Colloids and humic acids are negatively charged → stability

Adding coagulant (coagulation) → destabilisation

Flocculation → Growth of aggregates

After coagulation and flocculation removal of floc aggregates by sedimentation and/or filtration

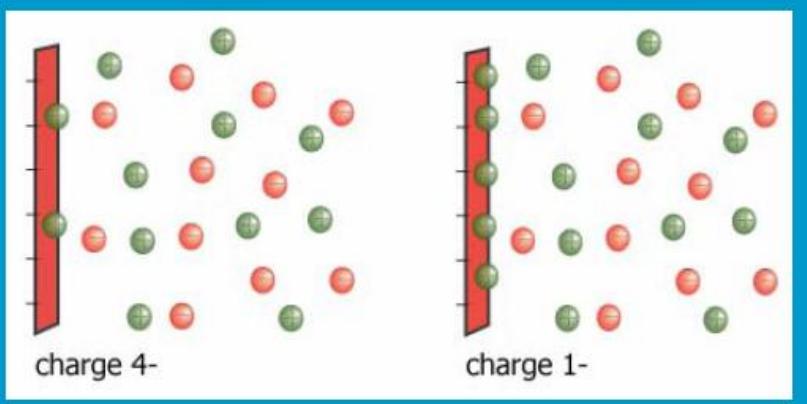







# Quality of surface water, worldwide

# Electrostatic coagulation



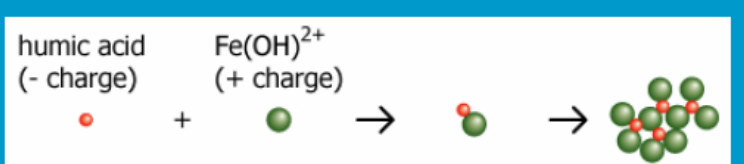

Dosage 0.025 mmol/l $Fe^{3+}$ → pH ≈ 3





# Adsorptive coagulation

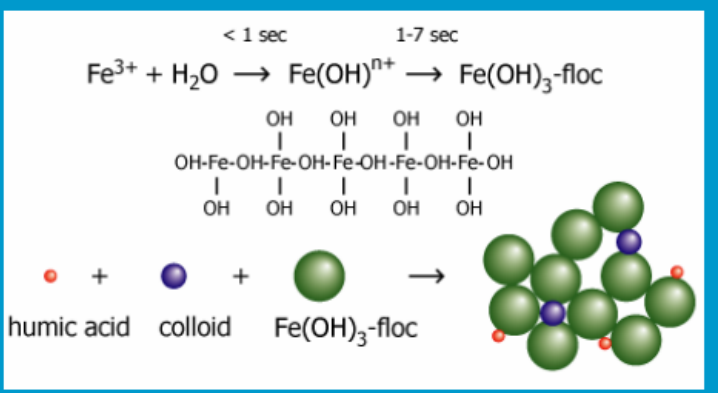
$Fe^{3+} + H_2O \xrightarrow{< 1\ sec} Fe(OH)^{2+}$




Adsorptive coagulation occurs at a low pH, because positive hydrolysis products are needed.

restabilisation:
- under-dosage of coagulants
- over-dosage of coagulants

# Precipitation (sweep) coagulation




At low turbidites
$Fe(OH)_3$ floc is neutral, flocs can collide





# Conclusions coagulation

electrostatic coagulation:
- → not of importance in drinking water treatment

adsorptive coagulation
- → colour at low pH, dosing proportional with removal of organic compounds
- → low dosing
- → high dosing results in re-stabilisation
- → optimum with low pH
- → mostly for colour (organic compounds)

precipitation coagulation
- → no re-stabilisation
- → high dosing
- → for turbidity removal
- → evident optimum pH 8 with iron
  pH 6 with aluminum

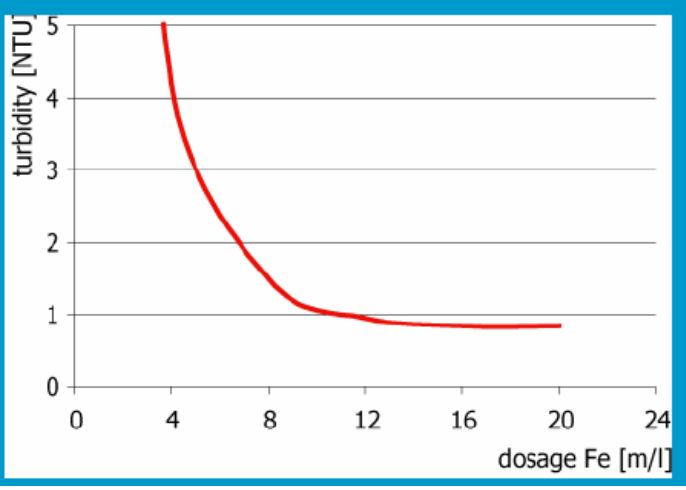
## Jar test apparatus

Variation in: pH, dosage, flocculation time, sedimentation time, stirring energy

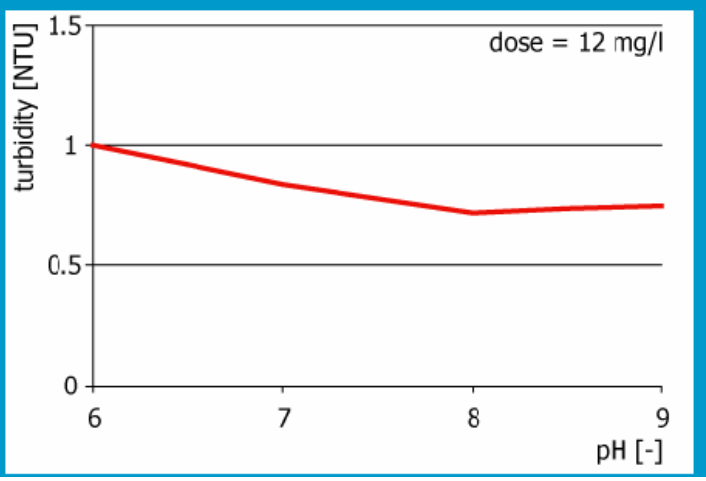




## Optmising dosage



## Optimising pH





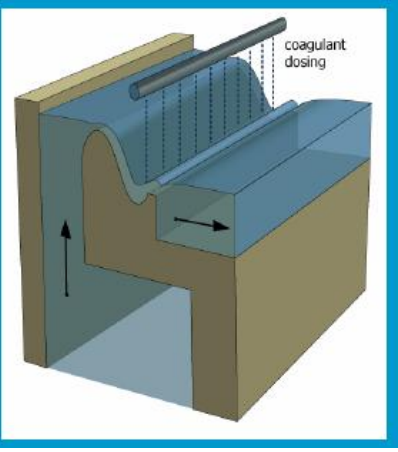



## Rapid mixing

- mechanical mixers
- static mixers

parameters:
- residence time (T)
- velocity gradient ($G_c$)

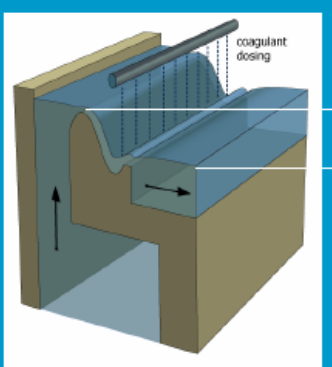
# Weir mixer








# Example weir mixing

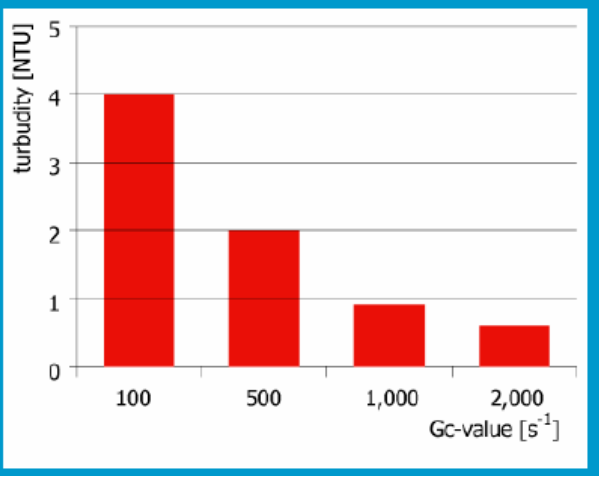



static mixer $G_c = \sqrt{\frac{\rho_w \cdot g \cdot \Delta H}{\tau_c \cdot \mu}}$

Example:
$\Delta H = 0.75$ m; $Q = 4500$ m³/h; $V = 2$ m³
$T = 20°C \rightarrow \mu = 1.01 \cdot 10^{-3}$ N·s/m²
$Q = 4500$ m³/h $= 1.25$ m³/s
$\tau_c = 2/1.25 = 1.60$ sec

$$G_c = \sqrt{\frac{1000 \cdot 9.81 \cdot 0.75}{1.01 \cdot 10^{-3} \cdot 1.6}} = 2134 s^{-1}$$

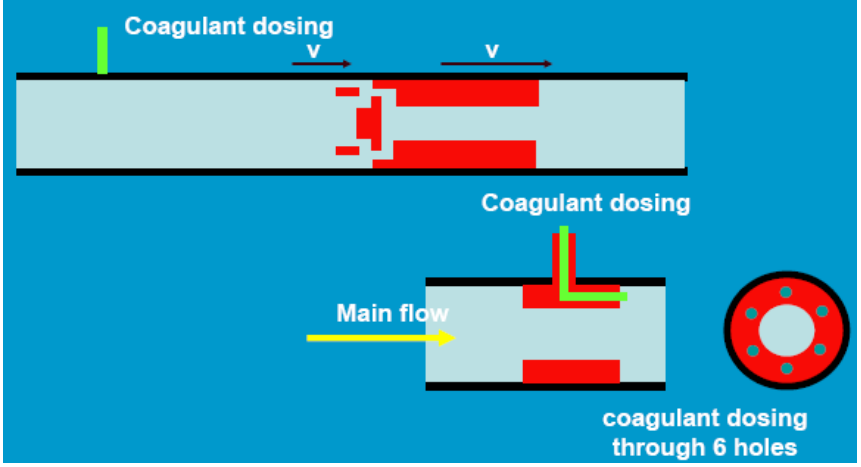
## Variation in velocity gradient $G_c$



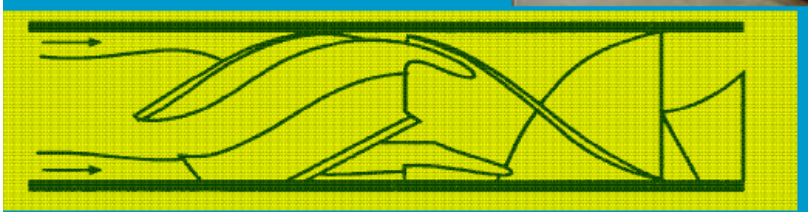





## Construction forms of static mixers



# Construction forms of static mixers







## Mechanical mixing

$$G = \sqrt{\frac{P}{\mu V}}$$

| | | |
|---|---|---|
| $G_c$ = | velocity gradient for rapid mixing | [$s^{-1}$] |
| P = | dissipated power | [W] |
| $\mu$ = | viscosity | [$N \cdot s/m^2$] |
| V = | volume of mixing tank | [$m^3$] |

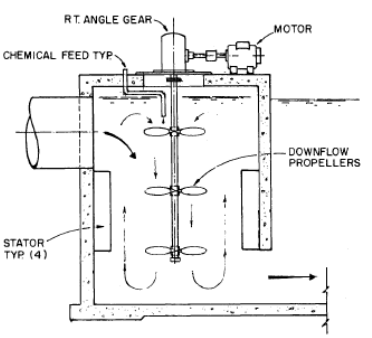
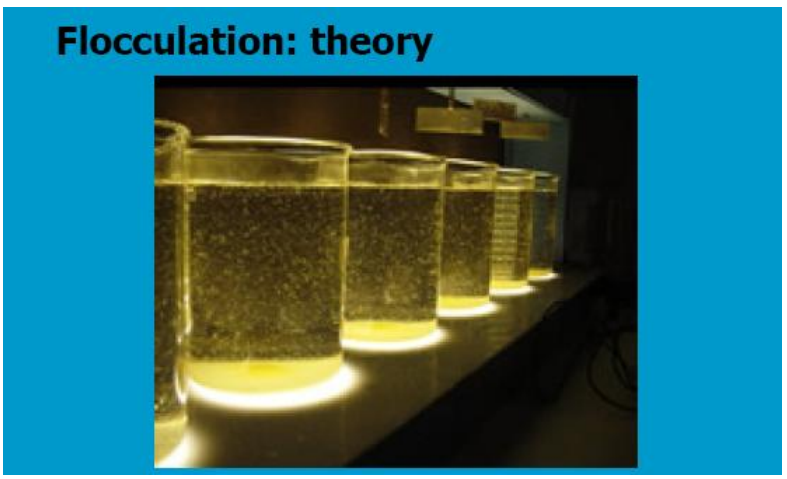



**FIGURE 6.1** Propeller-type mechanical flash mixer.

# Flocculation






# Mechanical flocculation

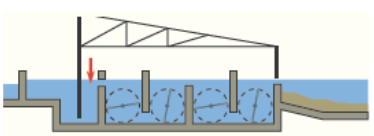


Figure 22 - Floc formation installation WRK I/II

# Hydraulic flocculation

Figure 25 - Hydraulic floc formation





# clariflocculators

## Velocity gradient

Energy input →
turbulence →
collision of particles → flocs

Degree of energy supply = velocity gradient

$$G_v = \sqrt{\frac{P}{V \bullet \eta}}$$

Velocity gradient in practice: 10-500$s^{-1}$
Considering floc break up: tapered flocculation





## العوامل المؤثرة على التنديف

- أولا الأس الهيدروجيني حيث يفضل أن تعمل الشبة في وسط يتراوح قيمة ال PH من ٥.٥ الي ٧.٥
- قيمة ال G value كما سق الذكر .
- جرعة الشبة المستخدمة ويتم تحديدها بواسطة ال jar test كما سياتي تفصيلا.
- عكارة المياه الخام ، حيث يصعب تنديف المياه قليلة العكارة

ضبط طلمبة حقن الشبة:-

**لضبط طلمبة حقن الشبة يجب أن تتوافر المعلومات الآتية:**

- **تركيز محلول الشبة ( ٢٠-٣٠% )**
- **كمية تصرف المياه الخام**
- **اقصى تصرف لطلمبة حقن الشبة**
- **الجرعة المراد إضافتها**

$$stroke = \frac{flowrate(m^3/hr) * dose(gm/m^3) * 100}{concentration * pump\max output * 1000}$$





## التأكد من جرعة الشبة

**الجرعة = ( ١٠٠ × فرق القلوية )/٤٥ ( في حالة الشبة الصلبة )**

**الجرعة = فرق القلوية × ٤.٤ ( في حالة الشبة السائلة )**

# الحكم على جودة عملية التنديف

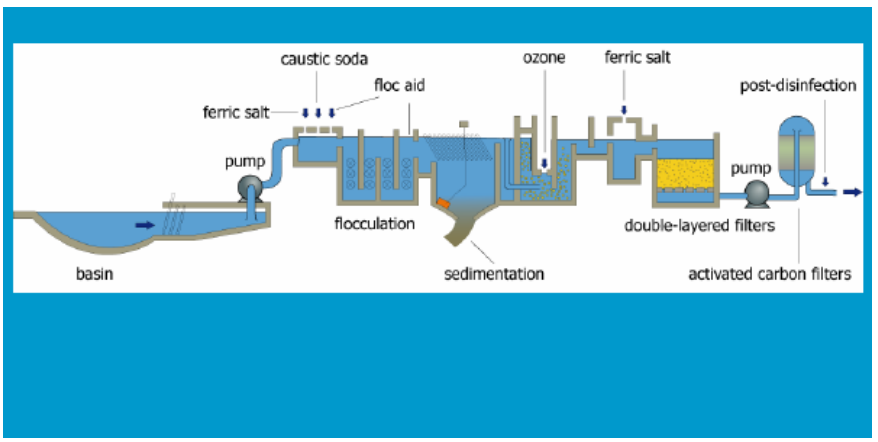
- معدل تكوين الندف
- شفافية المياه تحت و بين الندف
- حجم هذه الندف
- كمية الندف المتكونة
- معدل ترسيب الندف
- شفافية المياه للطبقة الرائقة





# تقنيات معالجة مياه الشرب

## عمليات المعالجة
## الترويق ( clarification):




## عملية الترويق في مخطط عمليات المعالجة



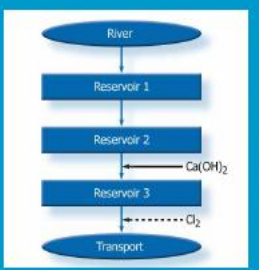


# أنواع الترسيب

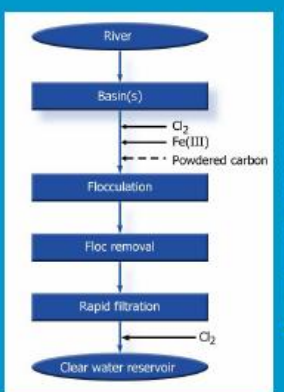
- الترسيب الطبيعي.
- الترسيب باستخدام المروبات.

# الترسيب الطبيعي

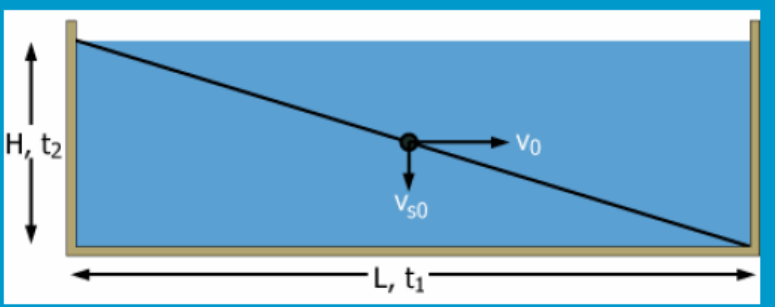
## Natural settling



## Settling after coagulation & flocculation



# Horizontal sedimentation



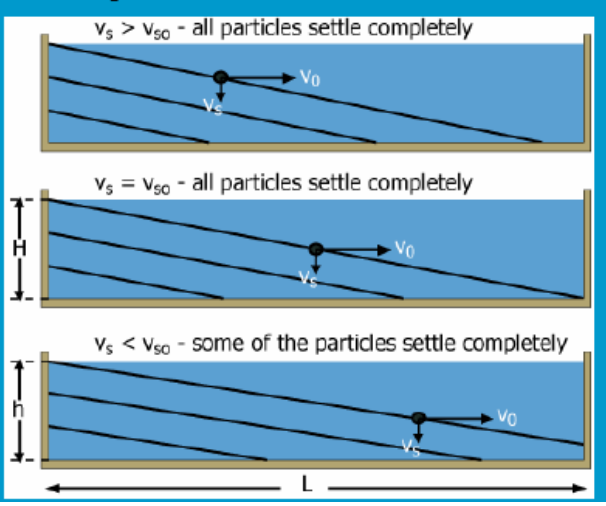

$$\frac{v_0}{L} \le \frac{v_{s0}}{H} \Rightarrow \frac{Q}{B \cdot H \cdot L} \le \frac{v_{s0}}{H} \Rightarrow \frac{Q}{B \cdot L} = v_{s0} \le v_s$$

Settling only depends on the surface loading $v_{s0}$, depth H has no influence





# Efficiency horizontal sedimentation



# العوامل المؤثرة علي عملية الترسيب

١. تركيز المواد العالقة في المياه
٢. شكل وحجم وكثافة المواد العالقة
٣. درجة حرارة الماء ودرجة لزوجته
٤. مدة بقاء الماء في الحوض



# أنواع أحواض الترسيب

الأحواض
المستديرة

# الأحواض المستطيلة

منطقة دخول المياه

## منطقة الترسيب





## منطقة خروج الماء

# العوامل المؤثرة في عملية الترسيب

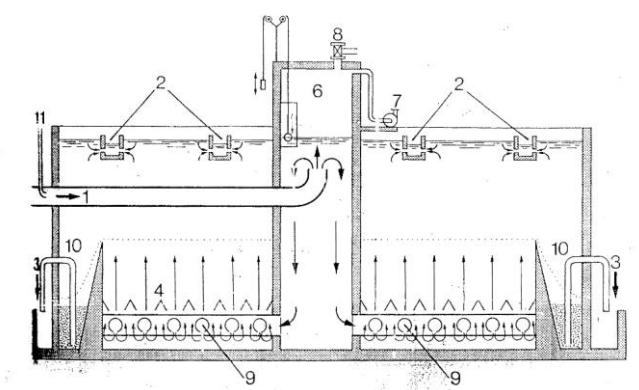
- ✓حجم الحبيبات
- ✓شكل الحبيبات
- ✓كثافة الحبيبات
- ✓درجة حرارة الماء
- ✓الشحنة الكهربائية للجسيمات
- ✓سرعة سريان الماء في الحوض
- ✓مدة بقاء الماء في الحوض (مدة المكث –Detention time)



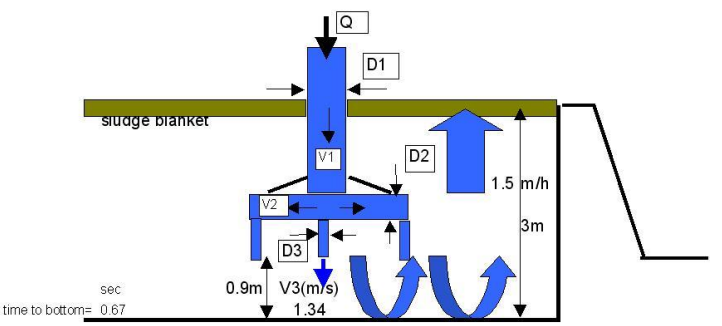


# الترويق باستخدام بطانية الروبة

المروق النابض



# مروقات الترايدنت



## التخلص من الروبة

**-:أسباب التخلص من الروبة**

**•منع تداخلها مع المياه المروقة وإعادة تعكيرها .**

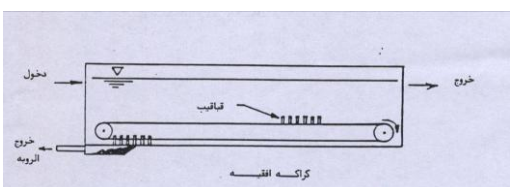
**•منع تكاثر البكتريا التي تسبب طعما ورائحة غير مقبولين للماء**

**•تفادي شغل حيز كبير من الحوض ، يخفض من كفاءة تشغيله**

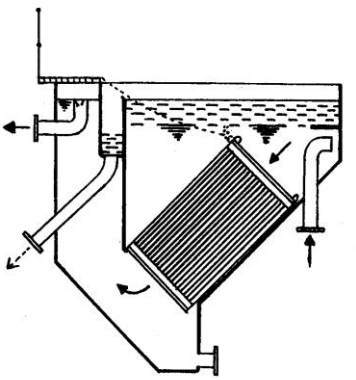
**مع انخفاض نوعية المياه المنتجة لانخفاض مدة المكث**

# أنظمة جمع الروبة

# تجميع الروبة في الأحواض المستطيلة



# أنظمة معالجة الروبة

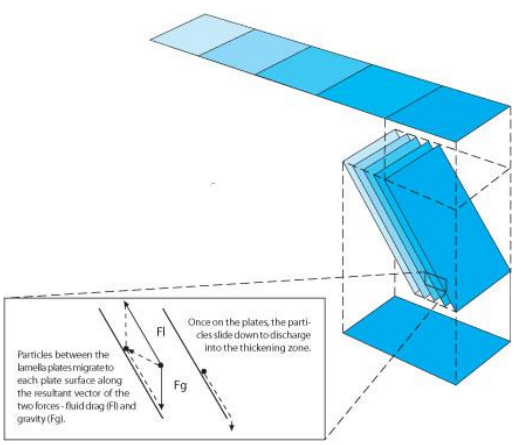






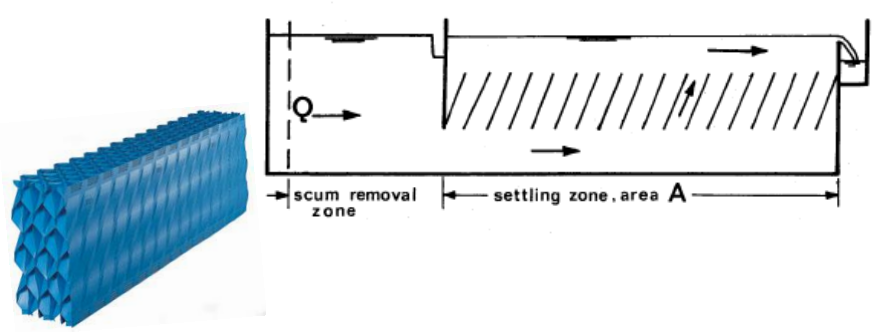
## تطوير مرحلة الترسيب

**ألواح اللاميلا**



## تطوير الأحواض المستطيلة

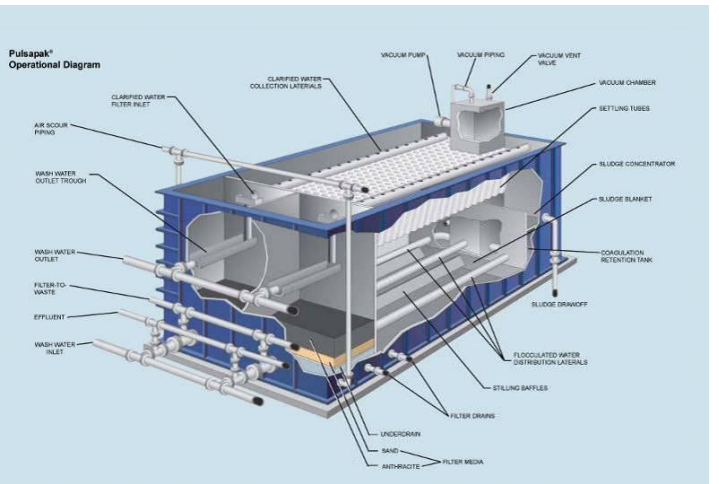
## تطوير الأحواض المستطيلة باستخدام أنابيب الترسيب



## تطوير المروقات الدائرية

*Installation of a Svedala LTC Combi lamella clarifier next to a conventional round thickener with approximately the same clarification area.*

## تطوير المروق النابض باستخدام ألواح اللاميلا





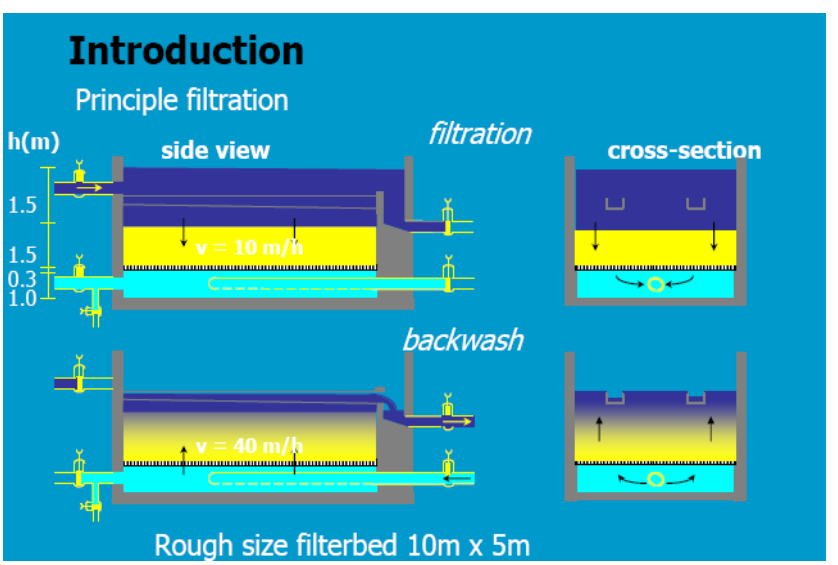



## مخططات التطوير أحد أهم دعائم نجاح عمليات المعالجة

# تقنيات معالجة مياه الشرب
# الترشيح Filtration

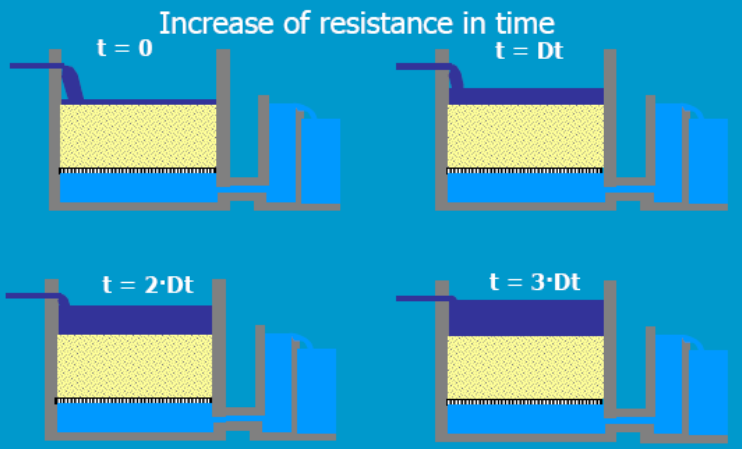






## Introduction

Filtration








# آلية الترشيح

## Filtration mechanisms

On top of filter bed

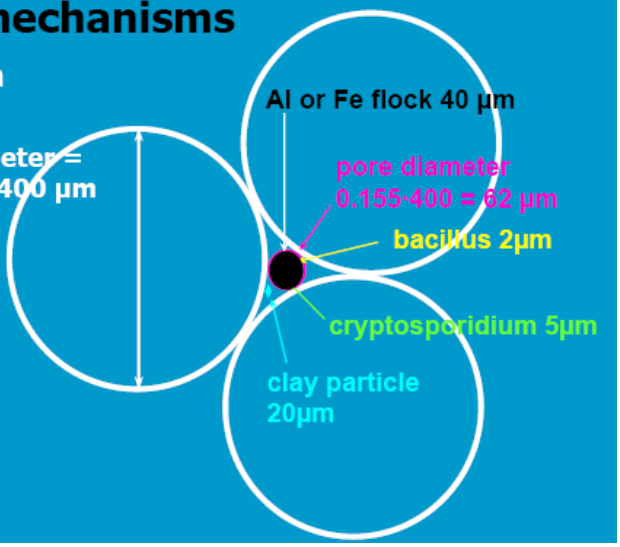
1. Sieve mechanism on top of the filterbed

In filter bed

2. transport of particles to grains and attachment to grains
3. chemical and biological transformations

## Filtration mechanisms

Sieve mechanism

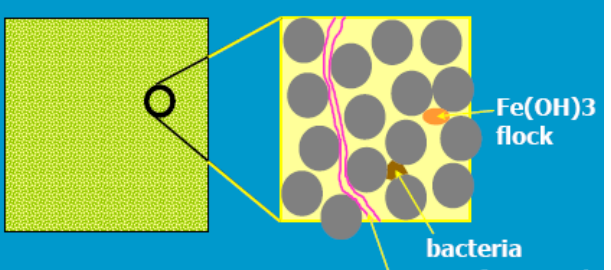







## Filtration mechanisms

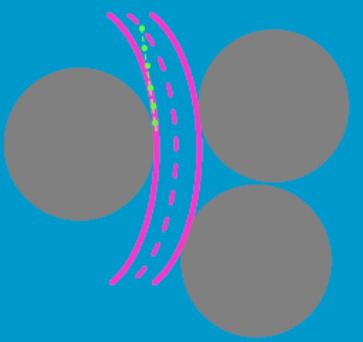
Transport and attachment




1. sedimentation
2. interception
3. diffusion
4. inertia
5. turbulance

## Filtration mechanisms

Sedimentation



trajectory
route

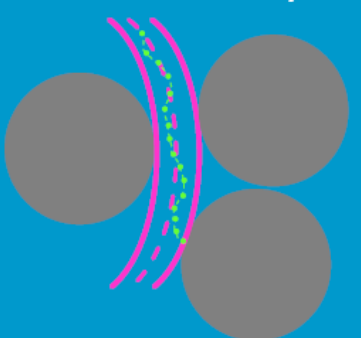




## Filtration mechanisms

Diffusion

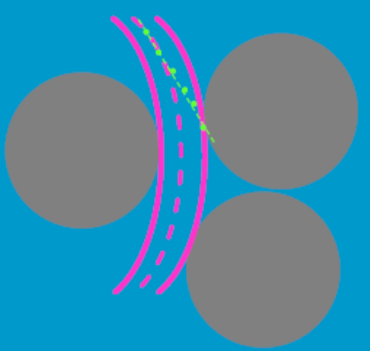
**Mostly small parts**



trajectory
route

## Filtration mechanisms

Inertia



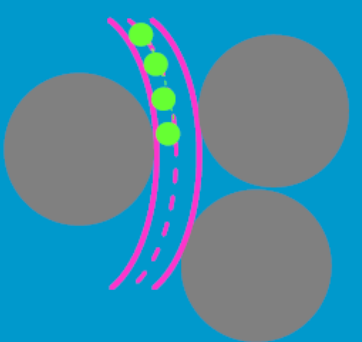
trajectory
route





## Filtration mechanisms

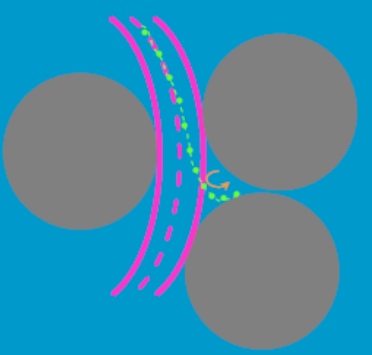
Interception



trajectory
route

## Filtration mechanisms

Turbulance



trajectory
route





## Filtration mechanisms

Chemical transformations

$4 \cdot Fe^{2+} + O_2 + 8 \cdot HCO_3^- \rightarrow 4 \cdot Fe(OH)_3 \downarrow + 8 \cdot H_3O^+$

$2 \cdot Mn^{2+} + O_2 + 6 \cdot H_2O \rightarrow 2 \cdot MnO_2 \downarrow + 4 \cdot H_3O^+$

Results in clogging of the filter

1 mg $Fe^{2+}$ uses 0.14 mg/l $O_2$

1 mg $Mn^{2+}$ uses 0.29 mg/l $O_2$

## Filtration mechanisms

Biological transformations

$2 \cdot NH_4^+ + 3 \cdot O_2 + 2 \cdot H_2O \rightarrow 2 \cdot NO_2^- + 4 \cdot H_3O^+$

$2 \cdot NO_2^- + O_2 \rightarrow 2 \cdot NO_3^-$

Nitrosomonas and nitrobacter

1 mg $NH_4^+$ uses 3.55 mg/l $O_2$

AOC, organic material

$CH_2O + O_2 \rightarrow CO_2 + H_2O$

Bacteria production:

| | |
|---|---|
| nitrosomonas | 0.15 g bacteria/ g ammonia |
| nitrobacter | 0.06 g bacteria/ g nitrite |
| methane | 0.10 g bacteria/ g methane |

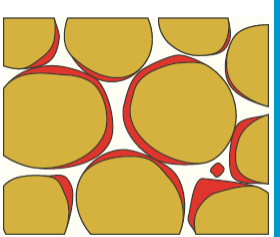
$CH_4 + 2 \cdot O_2 \rightarrow 2 \cdot H_2O + CO_2$

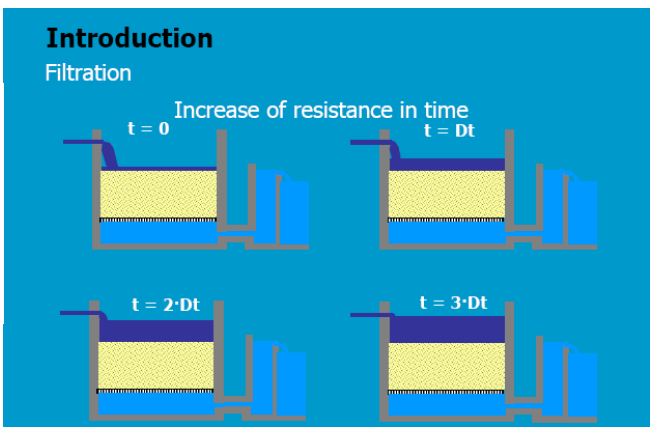
1 mg/l $CH_4$ uses 4 mg/l $O_2$

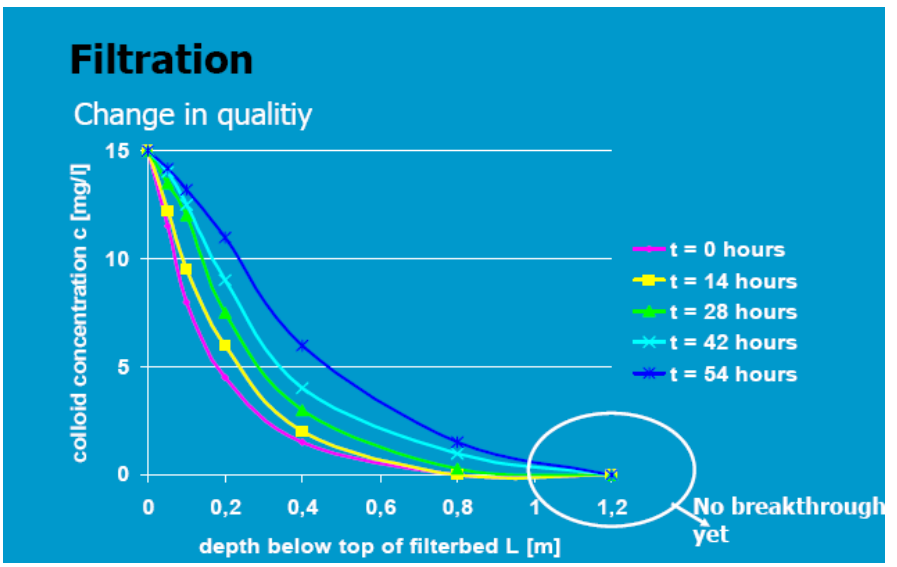




## كيف يعمل المرشح؟؟

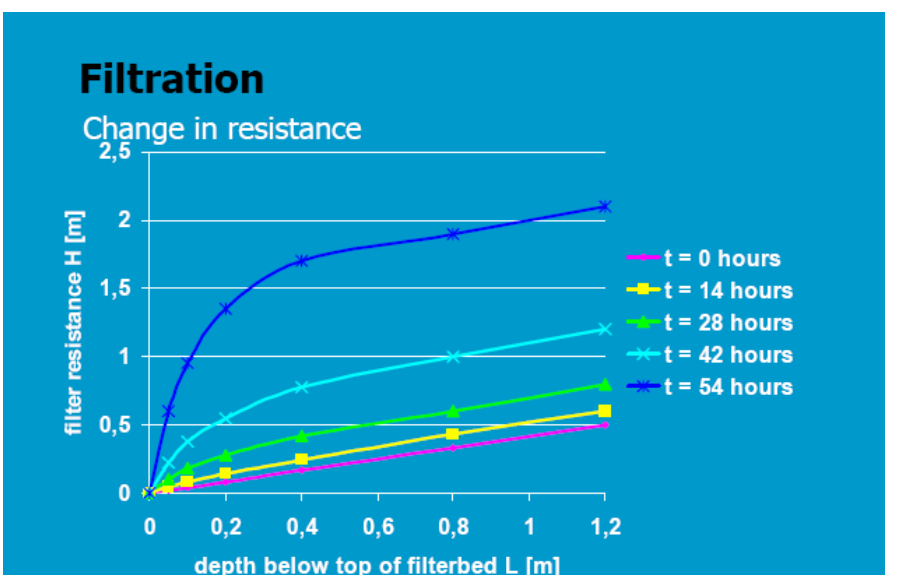





Filtration
Change in qualitiy
colloid concentration c [mg/l]
15
10
5
0
0
0,2
0,4
0,6
0,8
1
1,2
depth below top of filterbed L [m]
t = 0 hours
t = 14 hours
t = 28 hours
t = 42 hours
t = 54 hours
No breakthrough yet

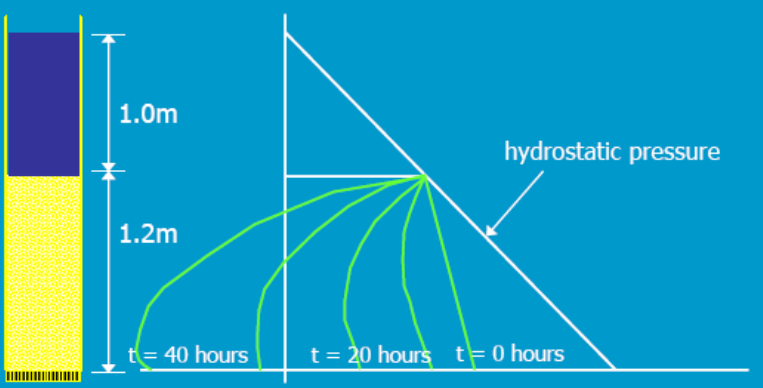





Filtration
Change in resistance
filter resistance H [m]
2,5
2
1,5
1
0,5
0
0
0,2
0,4
0,6
0,8
1
1,2
depth below top of filterbed L [m]
t = 0 hours
t = 14 hours
t = 28 hours
t = 42 hours
t = 54 hours

# Filtration

## Lindquist diagram








# Filtration

Filter run time

Filter runtime depends on:
- influent = Tq (= run time quality)
  chemical, physical and bacteriological composition of the water

-design parameters = Tr (= run time resistance)
  filtration speed, filter bed thickness, grain size, water level on top, grain size distribution, kind of filter material

conditions:
- Tq > Tr
- Tq, Tr > 1 day

# مواصفات الوسط الترشيحي

$$U = \frac{d_{60}}{d_{10}}$$

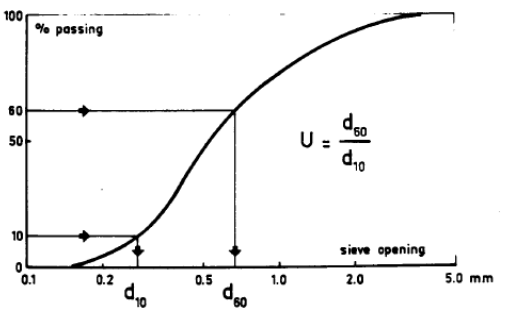
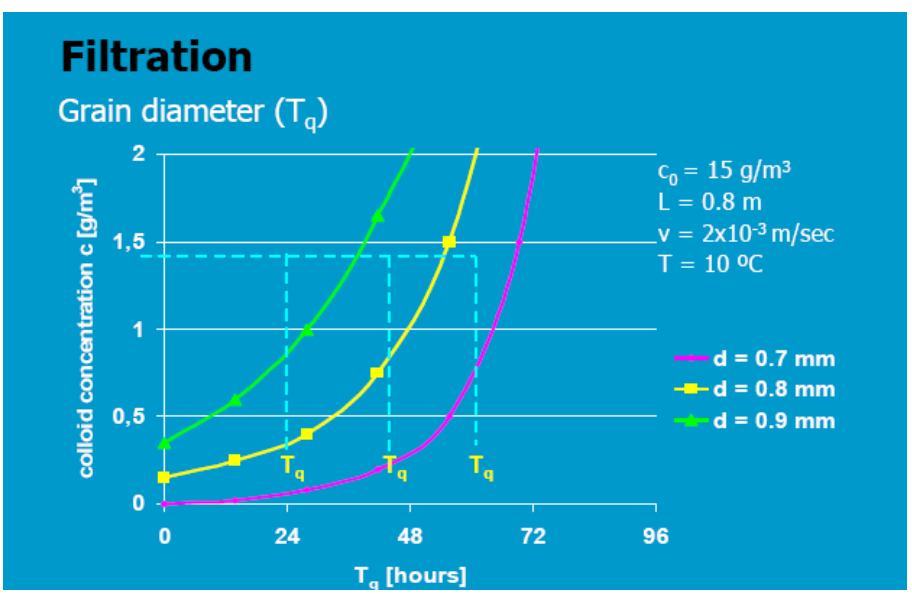
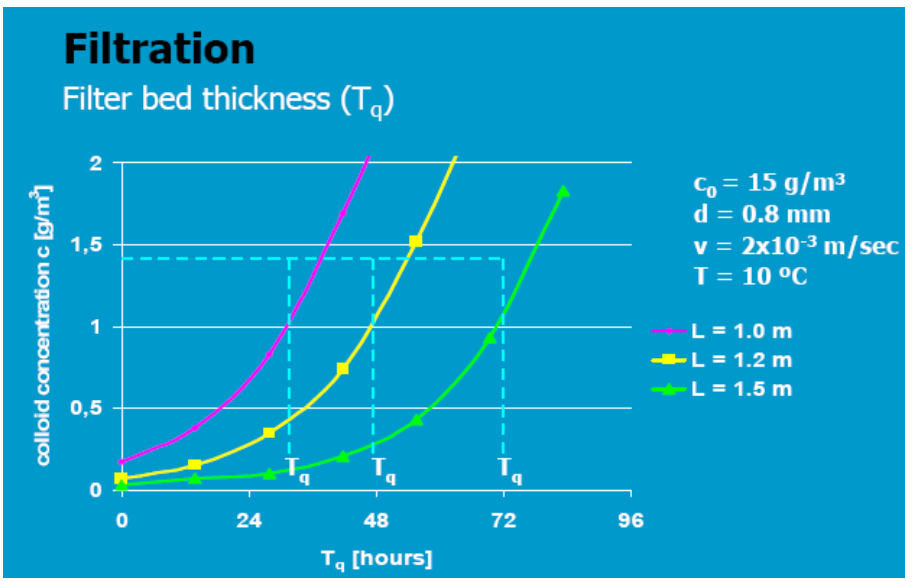



Fig. 2.23 Grain size analysis



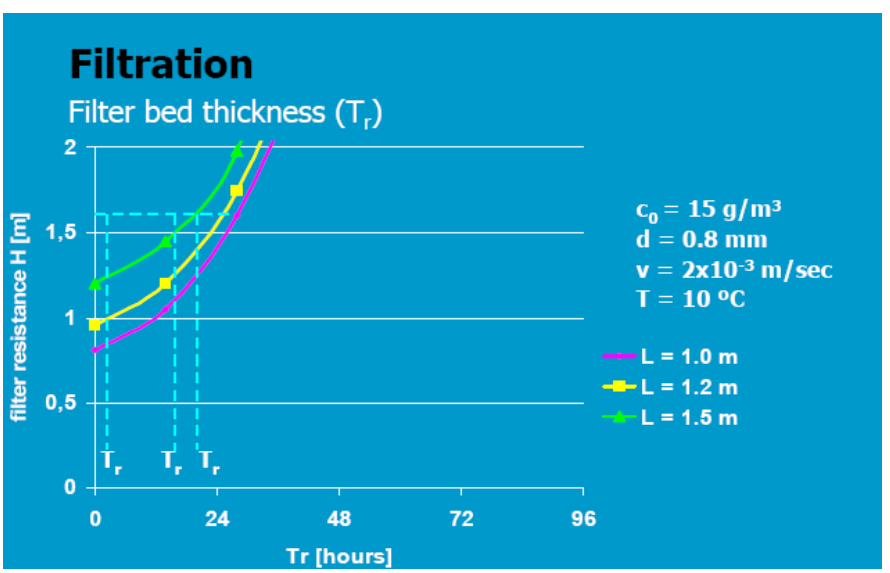





Filtration
Filter bed thickness ($T_q$)
colloid concentration c [g/m³]
2
1,5
1
0,5
0
0
24
48
72
96
$T_q$ [hours]
$T_q$
$T_q$
$T_q$
$c_0$ = 15 g/m³
d = 0.8 mm
v = 2x10⁻³ m/sec
T = 10 °C
L = 1.0 m
L = 1.2 m
L = 1.5 m






Filtration
Filter bed thickness ($T_r$)
filter resistance H [m]
2
1,5
1
0,5
0
0
24
48
72
96
Tr [hours]
$T_r$
$T_r$
$T_r$
$c_0$ = 15 g/m³
d = 0.8 mm
v = 2x10⁻³ m/sec
T = 10 °C
L = 1.0 m
L = 1.2 m
L = 1.5 m

## Filtration

### Influence of parameters

Influence parameters on $T_q$ and $T_r$

| | $T_q$ | $T_r$ |
|---|---|---|
| higher velocity v | shorter | shorter |
| larger bed thicknes L | longer | shorter |
| larger grain diameter d | shorter | longer |





**التطبيقات المختلفة للترشيح**

**المرشح متعدد الطبقات**

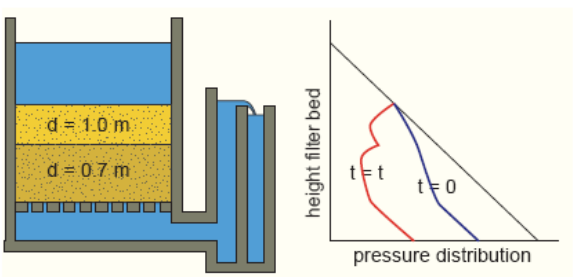



*Figure 23 - Resistance build-up for a multiple layer filter*

# الترشيح بالضغط

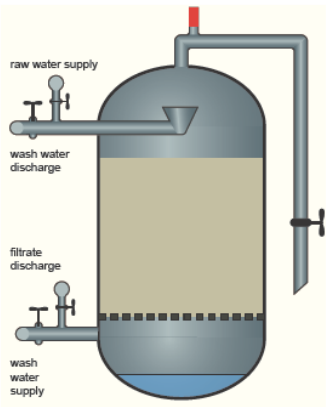



*Figure 24 - Principle of pressure filtration*





# الترشيح من أسفل إلي اعلي

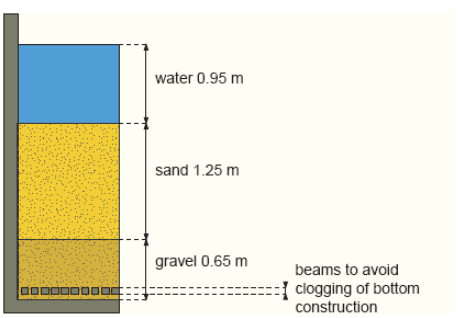



*Figure 28 - Principle of uplow filtration*

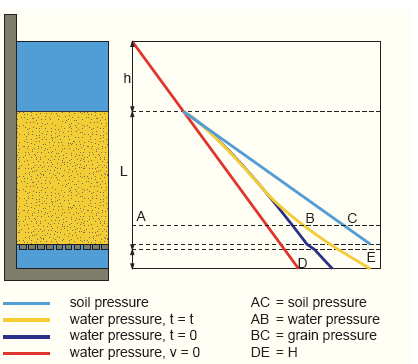



*Figure 29 - Pressure distribution in an upflow filter*

## لترشيح بواسطة الحجر الجيري ( limestone filtration )

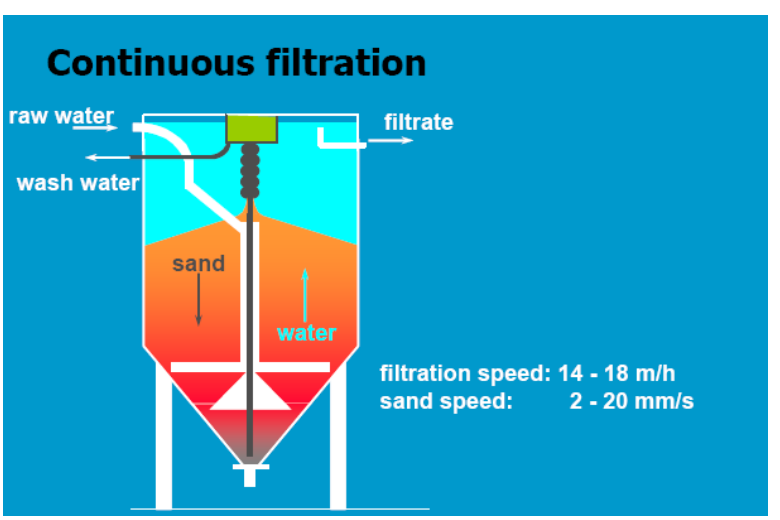
*Figure 4 - Aeration above limestone filter*

$$CaCO_3 + CO_2 + H_2O \longleftrightarrow Ca^{2+} + 2 \cdot HCO_3^-$$

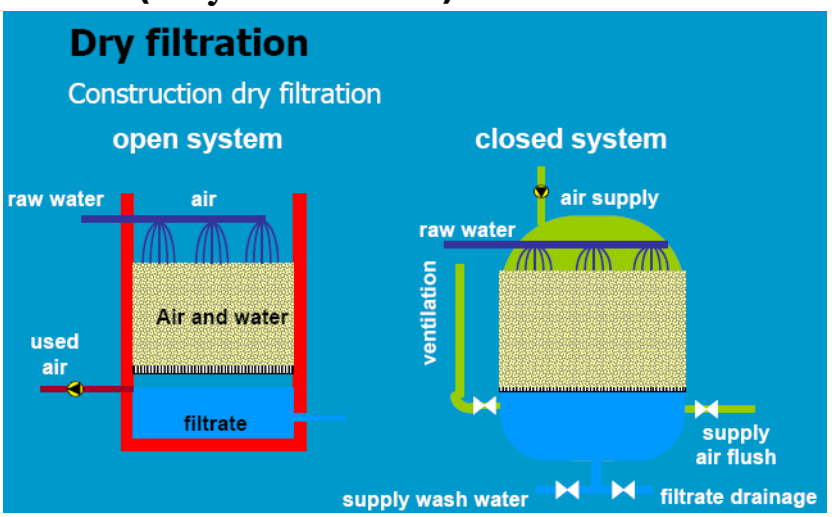




## الترشيح المتواصل



# المرشحات الجافة ( dry filtration )








# المرشحات الرملية البطيئة

# أنواع المرشحات الرملية السريعة

## يمكن تصنيف المرشحات تبعا :-

- عمق المنشأ الخرساني للمرشح حيث تنقسم المرشحات الي مرشحات عميقة ( deep filters ) و مرشحات ضحلة ( shallow filters ) وهو ما يدل علي كيفية التحكم في المرشح و دخوله أو إخراجه من الخدمة .
- يمكن تصنيف المرشح تبع لارتفاع المياه الخام فوق الوسط الترشيحي أثناء الترشيح فنقول ان المرشح ذو منسوب ثابت أو منسوب متغير .
- كما ينم تصنيف المرشح من حيث ثبات أو تغير معدل الترشيح فنقول مرشح ذو معدل ترشيح متناقص .





# أنظمة التحكم في المرشحات

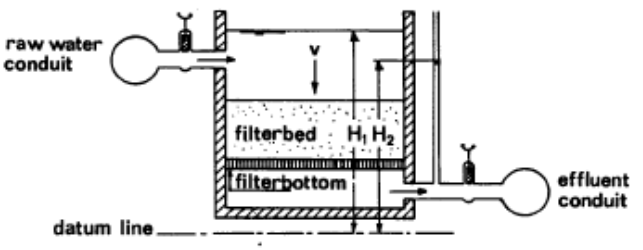
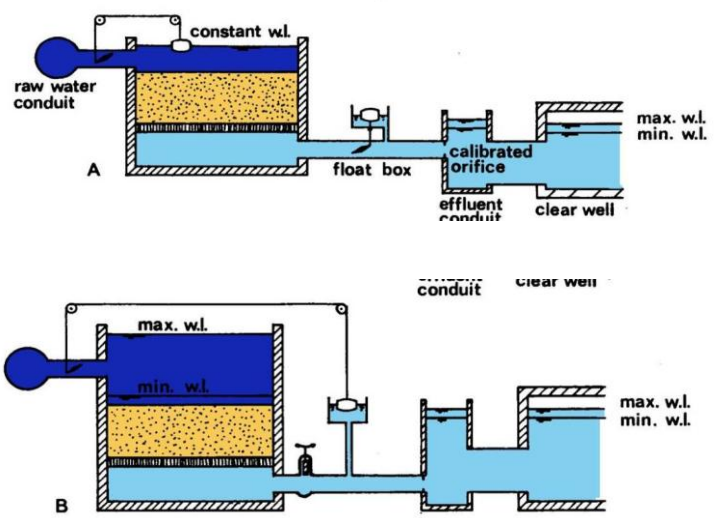
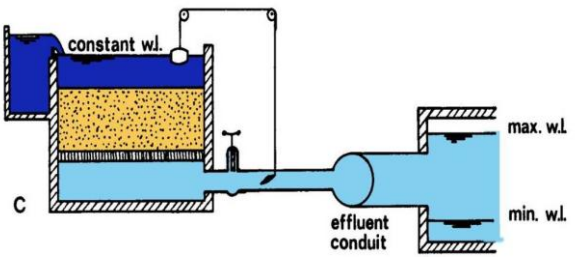


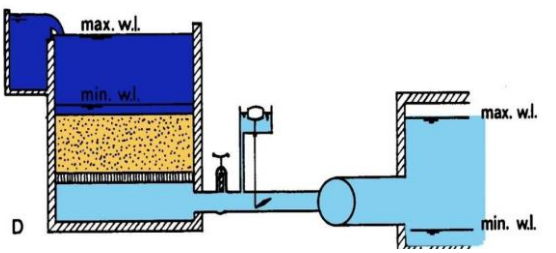

Fig. 4.8 Basic factors in filter control














## Backwash

| | |
|---|---|
| Backwash criteria: | $T_q$, $T_r$ |
| Goal: | removal of sludge, bacteria, increase of porosity |
| Means: | water, water and air |

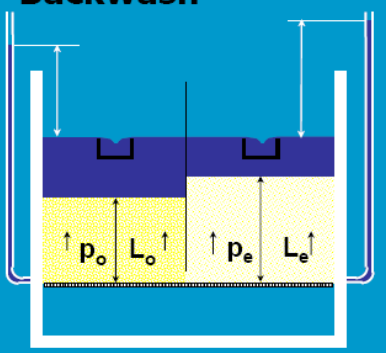
## إجراءات الغسيل

# الإجراءات والأهداف

| المرحلة | المادة المستخدمة في الغسيل | الهدف |
|---|---|---|
| ١ | هواء أو ماء | بدء دخول المادة المستخدمة في الغسيل في الاتجاه المعاكس |
| ٢ | هواء + ماء | عمل احتكاك لفصل الجزيئات العالقة ( scoring ) |
| ٣ | ماء يزداد تصرفة تدريجيا | طرد الهواء من الوسط الترشيحي |
| ٤ | ماء فقط | طرد المواد العالقة |
| ٥ | ماء يقل تصرفه تدريجيا | لتعود الجزيئات كل الي مكانة حسب الحجم والكثافة |

# ماذا يحدث أثناء الغسيل

## Backwash




low backwash velocity: no expansion

high backwash velocity: expansion

$$E = \frac{L_e - L_o}{L_o}$$

$\phi$ = 0.8 mm -> E = 15 - 20%

$\phi$ = 1.2 mm -> E = 10%

$$(1-p_o)\cdot L_o = (1-p_e)\cdot L_e$$

$$p_e = \frac{p_o + E}{1+E}$$





## Backwash

$$H = 130 \cdot \frac{\nu^{0.8}}{g} \cdot \frac{(1-p_e)^{1.8}}{p_e^{\,3}} \cdot \frac{v^{1.2}}{d^{1.8}} \cdot L_e$$

$$\rho_w \cdot g \cdot H_{max} = (1-p_o)\cdot L_o \cdot (\rho_f - \rho_w)\cdot g$$

$$H_{max} = (1-p_o)\cdot L_o \cdot \frac{\rho_f - \rho_w}{\rho_w}$$

with $p_o = 0.4$ en $\rho_f = 2600$ kg/m$^3$ then $H \approx L$

## Backwash

Rules of thumb for backwashing

velocities:

| | |
|---|---|
| water: | 7 - 30 m/h |
| air: | 30 - 90 m/h |

time laps:

| | |
|---|---|
| preliminary water flush: | 2 - 5 min |
| water and air flush: | 5 - 10 min |
| final water flush: | 5 - 10 min |

backwashvolume = 1 - 3% of produced water volume





## مشاكل المرشحات

# مشاكل المرشحات

*Figure 20 - Result of a poorly designed filter bottom*

الضغط السالب
كرات الطين
الشروخ
فوران الرمل
فقد الرمال
انسداد المرشح





# شكرا لحسن الاستماع

Deutsche Gesellschaft fur Technische Zusammenarbeit (GTZ) GmbH
Water & Wastewater Management Program
Holding Company for Water and Wastewater (HCWW),
Corniche El Nil, Water Treatment Plant-Road El Farag
Tel.: +2 02 245 98 405/411
Fax: +2 02 245 98 405/411
Website: www.gtz.de

الوكالة الألمانية للتعاون الفني
برنامج إدارة مياه الشرب و الصرف الصحي
الشركة القابضة لمياه الشرب و الصرف الصحي
كورنيش النيل محطة مياه روض الفرج
تليفون : ٤١١/٥٠٤ ٩٨ ٢٤٥ ٠٢ ٠٠
فاكس: ٤١١/٥٠٤ ٩٨ ٢٤٥ ٠٢ ٠٠
موقع إلكتروني: www.gtz.de