Roundabout Planning and Operation

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Credit: 5 PDH

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3 Planning

Chapter 3 Planning

Chapter 1 presented a range of roundabout categories, and suggested typical daily service volume thresholds below which four-leg roundabouts may be expected to operate, without requiring a detailed capacity analysis. Chapter 2 introduced roundabout performance characteristics, including comparisons with other intersection forms and control, which will be expanded upon in this chapter. This chapter covers the next steps that lead up to the decision to construct a roundabout with an approximate configuration at a specific location, preceding the detailed analysis and design of a roundabout. By confirming that there is good reason to believe that roundabout construction is feasible and that a roundabout offers a sensible method of accommodating the traffic demand, these planning activities make unnecessary the expenditure of effort required in subsequent chapters.

Planning determines whether a roundabout is even feasible, before expending the effort required in subsequent steps.

Planning for roundabouts begins with specifying a preliminary configuration. The configuration is specified in terms of the minimum number of lanes required on each approach and, thus, which roundabout category is the most appropriate basis for design: urban or rural, single-lane or double-lane roundabout. Given sufficient space, roundabouts can be designed to accommodate high traffic volumes. There are many additional levels of detail required in the design and analysis of a high-capacity, multi-lane roundabout that are beyond the scope of a planning level procedure. Therefore, this chapter focuses on the more common questions that can be answered using reasonable assumptions and approximations.

Feasibility analysis requires an approximation of some of the design parameters and operational characteristics. Some changes in these approximations may be necessary as the design evolves. A more detailed methodology for performing the operational evaluation and geometric design tasks is presented later in Chapters 4 and 6 of this guide, respectively.

Some of the assumptions and approximations used in planning may change as the design evolves, but are sufficient at this stage to answer many common questions.

3.1 Planning Steps

The following steps may be followed when deciding whether to implement a roundabout at an intersection:

- Step 1: Consider the context. What are there regional policy constraints that must be addressed? Are there site-specific and community impact reasons why a roundabout of any particular size would not be a good choice? (Section 3.2)
- Step 2: Determine a preliminary lane configuration and roundabout category based on capacity requirements (Section 3.3). Exhibit 3-1 will be useful for making a basic decision on the required number of lanes. If Exhibit 3-1 indicates that more than one lane is required on any approach, refer to Chapters 4 and 6 for the more detailed analysis and design procedures. Otherwise, proceed with the planning procedure.
- Step 3: Identify the selection category (Section 3.4). This establishes why a roundabout may be the preferred choice and determines the need for specific information.

- Step 4: Perform the analysis appropriate to the selection category. If the selection is to be based on operational performance, use the appropriate comparisons with alternative intersections (Section 3.5).
- Step 5: Determine the space requirements. Refer to Section 3.6 and Appendix B for the right-of-way widths required to accommodate the inscribed circle diameter. Determine the space feasibility. Is there enough right-of-way to build it? This is a potential rejection point. There is no operational reason to reject a roundabout because of the need for additional right-of-way; however, right-of-way acquisition introduces administrative complications that many agencies would prefer to avoid.
- Step 6: If additional space must be acquired or alternative intersection forms are viable, an economic evaluation may be useful (Section 3.7).

The results of the steps above should be documented to some extent. The level of detail in the documentation will vary among agencies and will generally be influenced by the size and complexity of the roundabout. A roundabout selection study report may include the following elements:

Suggested contents of a roundabout selection study report.

- It may identify the selection category that specifies why a roundabout is the logical choice at this intersection;
- It may identify current or projected traffic control or safety problems at the intersection if the roundabout is proposed as a solution to these problems;
- It may propose a configuration, in terms of number of lanes on each approach;
- It may demonstrate that the proposed configuration can be implemented feasibly and that it will provide adequate capacity on all approaches; and
- It may identify all potential complicating factors, assess their relevance to the location, and identify any mitigation efforts that might be required.

Agencies that require a more complete or formal rationale may also include the following additional considerations:

- It may demonstrate institutional and community support indicating that key institutions (e.g., police, fire department, schools, etc.) and key community leaders have been consulted;
- It may give detailed performance comparisons of the roundabout with alternative control modes;
- It may include an economic analysis, indicating that a roundabout compares favorably with alternative control modes from a benefit-cost perspective; and
- It may include detailed appendices containing traffic volume data, signal, or all-way stop control (AWSC) warrant analysis, etc.

None of these elements should be construed as an absolute requirement for documentation. The above list is presented as a guide to agencies who choose to prepare a roundabout study report.

3.2 Considerations of Context

3.2.1 Decision environments

There are three somewhat different policy environments in which a decision may be made to construct a roundabout at a specific location. While the same basic analysis tools and concepts apply to all of the environments, the relative importance of the various aspects and observations may differ, as may prior constraints that are imposed at higher policy levels.

Will the roundabout be...
- **Part of a new roadway?**
- **The first in an area?**
- **A retrofit of an existing intersection?**

A new roadway system: Fewer constraints are generally imposed if the location under consideration is not a part of an existing roadway system. Right-of-way is usually easier to acquire or commit. Other intersection forms also offer viable alternatives to roundabouts. There are generally no field observations of site-specific problems that must be addressed. This situation is more likely to be faced by developers than by public agencies.

The first roundabout in an area: The first roundabout in any geographic area requires an implementing agency to perform due diligence on roundabouts regarding their operational and design aspects, community impacts, user needs, and public acceptability. On the other hand, a successfully implemented roundabout, especially one that solves a perceived problem, could be an important factor in gaining support for future roundabouts at locations that could take advantage of the potential benefits that roundabouts may offer. Some important considerations for this decision environment include:

- Effort should be directed toward gaining community and institutional support for the selection of a site for the first roundabout in an area. Public acceptance for roundabouts, like any new roadway facility, require agency staff to understand the potential issues and communicate these effectively with the impacted community;
- An extensive justification effort may be necessary to gain the required support;
- A cautious and conservative approach may be appropriate; careful consideration should be given to conditions that suggest that the benefits of a roundabout might not be fully realized. Collecting data on current users of the facility can provide important insights regarding potential issues and design needs;
- A single-lane roundabout in the near-term is more easily understood by most drivers and therefore may have a higher probability of acceptance by the motoring public;
- The choice of design and analysis procedures could set a precedent for future roundabout implementation; therefore, the full range of design and analysis alternatives should be explored in consultation with other operating agencies in the region; and
- After the roundabout is constructed, evaluating its operation and the public response could provide documentation to support future installations.

The first roundabout in an area requires greater education and justification efforts. Single-lane roundabouts will be more easily understood initially than multilane roundabouts.

Retrofit to an existing intersection in an area where roundabouts have already gained acceptance: This environment is one in which a solution to a site-specific problem is being sought. Because drivers are familiar with roundabout operation, a less intensive process may suffice. Double-lane roundabouts could be considered, and the regional design and evaluation procedures should have already been agreed upon. The basic objectives of the selection process in this case are to demonstrate the community impacts and that a roundabout will function properly during the peak period within the capacity limits imposed by the space available; and to decide whether one is the preferred alternative. If the required configuration involves additional right-of-way, a more detailed analysis will probably be necessary, using the methodology described in Chapter 4.

Many agencies that are contemplating the construction of their first roundabout are naturally reluctant to introduce complications, such as double-lane, yield-controlled junctions, which are not used elsewhere in their jurisdiction. It is also a common desire to avoid intersection designs that require additional right-of-way, because of the effort and expense involved in right-of-way acquisition. Important questions to be addressed in the planning phase are therefore:

- Will a minimally configured roundabout (i.e., single-lane entrances and circulatory roadway) provide adequate capacity and performance for all users, or will additional lanes be required on some legs or at some future time?
- Can the roundabout be constructed within the existing right-of-way, or will it be necessary to acquire additional space beyond the property lines?
- Can a single-lane roundabout be upgraded in the future to accommodate growth?

If not, a roundabout alternative may require that more rigorous analysis and design be conducted before a decision is made.

3.2.2 Site-specific conditions

Some conditions may preclude a roundabout at a specific location. Certain site-related factors may significantly influence the design and require a more detailed investigation of some aspects of the design or operation. A number of these factors (many of which are valid for any intersection type) are listed below:

Site-specific factors that may significantly influence a roundabout's design.

- Physical or geometric complications that make it impossible or uneconomical to construct a roundabout. These could include right-of-way limitations, utility conflicts, drainage problems, etc.
- Proximity of generators of significant traffic that might have difficulty negotiating the roundabout, such as high volumes of oversized trucks.
- Proximity of other traffic control devices that would require preemption, such as railroad tracks, drawbridges, etc.
- Proximity of bottlenecks that would routinely back up traffic into the roundabout, such as over-capacity signals, freeway entrance ramps, etc. The successful operation of a roundabout depends on unimpeded flow on the circulatory roadway. If traffic on the circulatory roadway comes to a halt, momentary intersection gridlock can occur. In comparison, other control types may continue to serve some movements under these circumstances.
- Problems of grades or unfavorable topography that may limit visibility or complicate construction.
- Intersections of a major arterial and a minor arterial or local road where an unacceptable delay to the major road could be created. Roundabouts delay and deflect all traffic entering the intersection and could introduce excessive delay or speed inconsistencies to flow on the major arterial.

- Heavy pedestrian or bicycle movements in conflict with high traffic volumes. (These conflicts pose a problem for all types of traffic control. There is very little experience on this topic in the U.S., mostly due to a lack of existing roundabout sites with heavy intermodal conflicts).
- Intersections located on arterial streets within a coordinated signal network. In these situations, the level of service on the arterial might be better with a signalized intersection incorporated into the system. Chapter 8 deals with system considerations for roundabouts.

The existence of one or more of these conditions does not necessarily preclude the installation of a roundabout. Roundabouts have, in fact, been built at locations that exhibit nearly all of the conditions listed above. Such factors may be resolved in several ways:

- They may be determined to be insignificant at the specific site;
- They may be resolved by operational modeling or specific design features that indicate that no significant problems will be created;
- They may be resolved through coordination with and support from other agencies, such as the local fire department; and
- In some cases, specific mitigation actions may be required.

All complicating factors should be resolved prior to the choice of a roundabout as the preferred intersection alternative.

The effect of a particular factor will often depend on the degree to which roundabouts have been implemented in the region. Some conditions would not be expected to pose problems in areas where roundabouts are an established form of control that is accepted by the public. On the other hand, some conditions, such as heavy pedestrian volumes, might suggest that the installation of a roundabout be deferred until this control mode has demonstrated regional acceptance. Most agencies have an understandable reluctance to introduce complications at their first roundabout.

3.3 Number of Entry Lanes

A basic question that needs to be answered is how many entry lanes a roundabout would require to serve the traffic demand. The capacity of a roundabout is clearly a critical parameter and one that should be checked at the outset of any feasibility study. Chapter 4 offers a detailed capacity computation procedure, mostly based on experiences in other countries. Some assumptions and approximations have been necessary in this chapter to produce a planning-level approach for deciding whether or not capacity is sufficient.

Since this is the first of several planning procedures to be suggested in this chapter, some discussion of the assumptions and approximations is appropriate. First, traffic volumes are generally represented for planning purposes in terms of Average Daily Traffic (ADT), or Average Annual Daily Traffic (AADT). Traffic operational analyses must be carried out at the design hour level. This requires an assumption of a K factor and a D factor to indicate, respectively, the proportion of the AADT assigned to the design hour, and the proportion of the two-way traffic that is assigned to the peak direction. All of the planning-level procedures offered in this chapter were based on reasonably typical assumed values for K of 0.1 and D of 0.58.

There are two site-specific parameters that must be taken into account in all computations. The first is the proportion of traffic on the major street. For roundabout planning purposes, this value was assumed to lie between 0.5 and 0.67. All analyses assumed a four-leg intersection. The proportion of left turns must also be considered, since left turns affect all traffic control modes adversely. For the purposes of this chapter, a reasonably typical range of left turns were examined. Right turns were assumed to be 10 percent in all cases. Right turns are included in approach volumes and require capacity, but are not included in the circulating volumes downstream because they exit before the next entrance.

The volume-to-capacity ratio of any roundabout leg is recommended not to exceed 0.85.

The capacity evaluation is based on values of entering and circulating traffic volumes as described in Chapter 4. The AADT that can be accommodated is conservatively estimated as a function of the proportion of left turns, for cross-street volume proportions of 50 percent and 67 percent. For acceptable roundabout operation, many sources advise that the volume-to-capacity ratio on any leg of a roundabout not exceed 0.85 (1, 2). This assumption was used in deriving the AADT maximum service volume relationship.

3.3.1 Single- and double-lane roundabouts

The resulting maximum service volumes are presented in Exhibit 3-1 for a range of left turns from 0 to 40 percent of the total volume. This range exceeds the normal expectation for left turn proportions. This procedure is offered as a simple, conservative method for estimating roundabout lane requirements. If the 24-hour volumes fall below the volumes indicated in Exhibit 3-1, a roundabout should have no operational problems at any time of the day. It is suggested that a reasonable approximation of lane requirements for a three-leg roundabout may be obtained using 75 percent of the service volumes shown on Exhibit 3-1.

If the volumes exceed the threshold suggested in Exhibit 3-1, a single-lane or double-lane roundabout may still function quite well, but a closer look at the actual turning movement volumes during the design hour is required. The procedures for such analysis are presented in Chapter 4.

3.3.2 Mini-roundabouts

Mini-roundabouts are distinguished from traditional roundabouts primarily by their smaller size and more compact geometry. They are typically designed for negotiation speeds of 25 km/h (15 mph). Inscribed circle diameters generally vary from 13 m to 25 m (45 ft to 80 ft). Mini-roundabouts are usually implemented with safety in mind, as opposed to capacity. Peak-period capacity is seldom an issue, and most mini-roundabouts operate on residential or collector streets at demand levels well below their capacity. It is important, however, to be able to assess the capacity of any proposed intersection design to ensure that the intersection would function properly if constructed.

At very small roundabouts, it is reasonable to assume that each quadrant of the circulatory roadway can accommodate only one vehicle at a time. In other words, a vehicle may not enter the circulatory roadway unless the quadrant on both sides of the approach is empty. Given a set of demand volumes for each of the 12 standard movements at a four-leg roundabout, it is possible to simulate the roundabout to estimate the maximum service volumes and delay for each approach. By making assumptions about the proportion of left turns and the proportion of cross street traffic, a general estimate of the total entry maximum service volumes of the roundabout can be made, and is provided in Exhibit 3-2. AADT maximum service volumes are represented based on an assumed K value of 0.10. Note that these volumes range from slightly more than 12,000 to slightly less than 16,000 vehicles per day. The maximum throughput is achieved with an equal proportion of vehicles on the major and minor roads, and with low proportions of left turns.

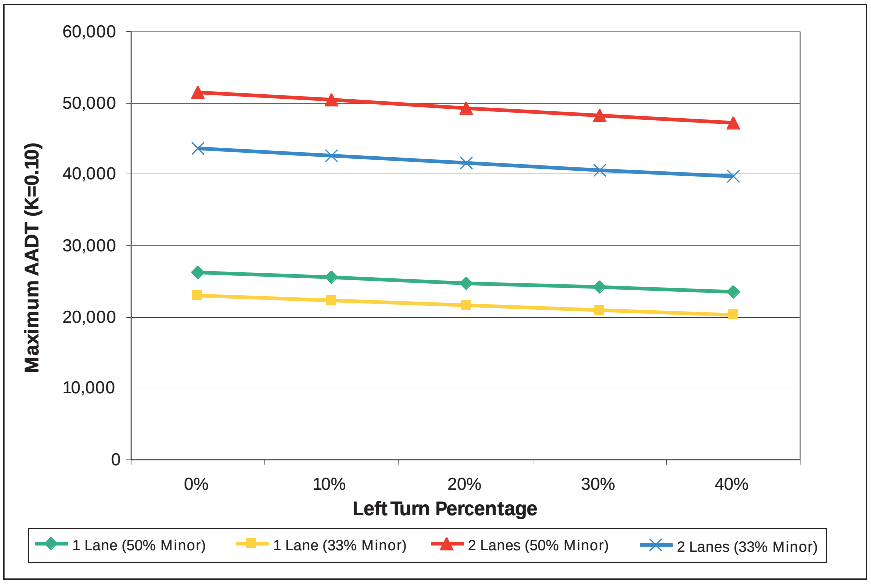

Exhibit 3-1. Maximum daily service volumes for a four-leg roundabout.

For three-leg roundabouts, use 75 percent of the maximum AADT volumes shown.

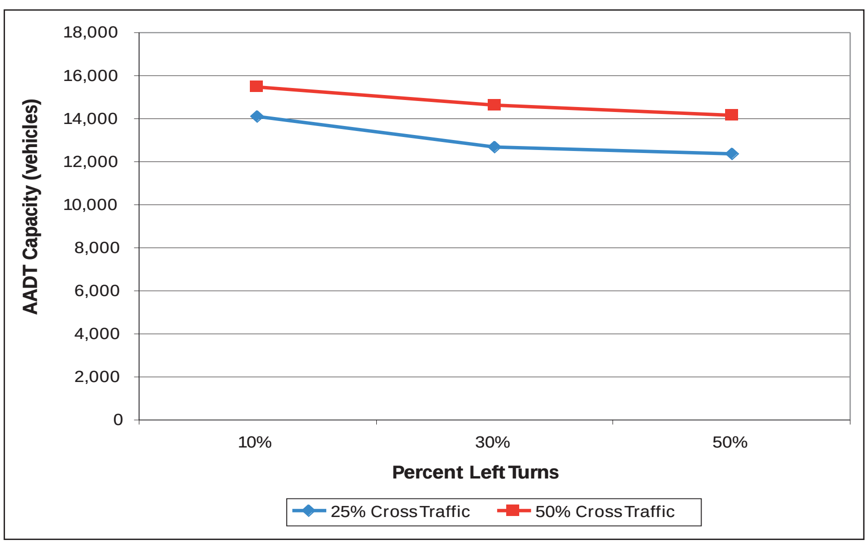

Exhibit 3-2. Planning-level maximum daily service volumes for mini-roundabouts.

3.4 Selection Categories

There are many locations at which a roundabout could be selected as the preferred traffic control mode. There are several reasons why this is so, and each reason creates a separate selection category. Each selection category, in turn, requires different information to demonstrate the desirability of a roundabout. The principal selection categories will be discussed in this section, along with their information requirements.

A wide range of roundabout policies and evaluation practices exists among operating agencies within the U.S. For example, the Florida Department of Transportation requires a formal "justification report" to document the selection of a roundabout as the most appropriate traffic control mode at any intersection on their State highway system. On the other hand, private developers may require no formal rationalization of any kind. It is interesting to note that the Maryland Department of Transportation requires consideration of a roundabout as an alternative at all intersections proposed for signalization.

It is reasonable that the decision to install a roundabout should require approximately the same level of effort as the alternative control mode. In other words, if a roundabout is proposed as an alternative to a traffic signal, then the analysis effort should be approximately the same as that required for a signal. If the alternative is stop sign control, then the requirements could be relaxed.

The following situations present an opportunity to demonstrate the desirability of installing a roundabout at a specific location.

3.4.1 Community enhancement

Roundabouts have been proposed as a part of a community enhancement project and not as a solution to capacity problems. Such projects are often located in commercial and civic districts, as a gateway treatment to convey a change of environment and to encourage traffic to slow down. Traffic volumes are typically well below the thresholds shown in Exhibit 3-1; otherwise, one of the more operationally oriented selection categories would normally be more appropriate.

The planning focus for community enhancement roundabouts should be to demonstrate that they will not create traffic problems that do not now exist.

Roundabouts proposed for community enhancement require minimal analysis as a traffic control device. The main focus of the planning procedure should be to demonstrate that they would not introduce traffic problems that do not exist currently. Particular attention should be given to any complications that would imply either operational or safety problems. The urban compact category may be the most appropriate roundabout for such applications. Exhibit 3-3 provides an example of a roundabout installed primarily for community enhancement.

3.4.2 Traffic calming

The decision to install a roundabout for traffic calming purposes should be supported by a demonstrated need for traffic calming along the intersecting roadways. Most of the roundabouts in this category will be located on local roads. Examples of conditions that might suggest a need for traffic calming include:

Conditions that traffic calming roundabouts may address.

- Documented observations of speeding, high traffic volumes, or careless driving activities;

Naples, FL

Exhibit 3-3. Example of community enhancement roundabout.

- Inadequate space for roadside activities, or a need to provide slower, safer conditions for non-automobile users; or
- New construction (road opening, traffic signal, new road, etc.) which would potentially increase the volumes of "cut-through" traffic.

Capacity should be an issue when roundabouts are installed for traffic calming purposes only because traffic volumes on local streets will usually be well below the level that would create congestion. If this is not the case, another primary selection category would probably be more suitable. The urban mini-roundabout or urban compact roundabout are most appropriate for traffic calming purposes. Exhibit 3-4 provides an example of roundabouts installed primarily for traffic calming.

3.4.3 Safety improvement

The decision to install a roundabout as a safety improvement should be based on a demonstrated safety problem of the type susceptible to correction by a roundabout. A review of crash reports and the type of accidents occurring is essential. Examples of safety problems include:

Safety issues that roundabouts may help correct.

- High rates of crashes involving conflicts that would tend to be resolved by a roundabout (right angle, head-on, left/through, U-turns, etc.);
- High crash severity that could be reduced by the slower speeds associated with roundabouts;

Exhibit 3-4. Example of traffic calming roundabouts.

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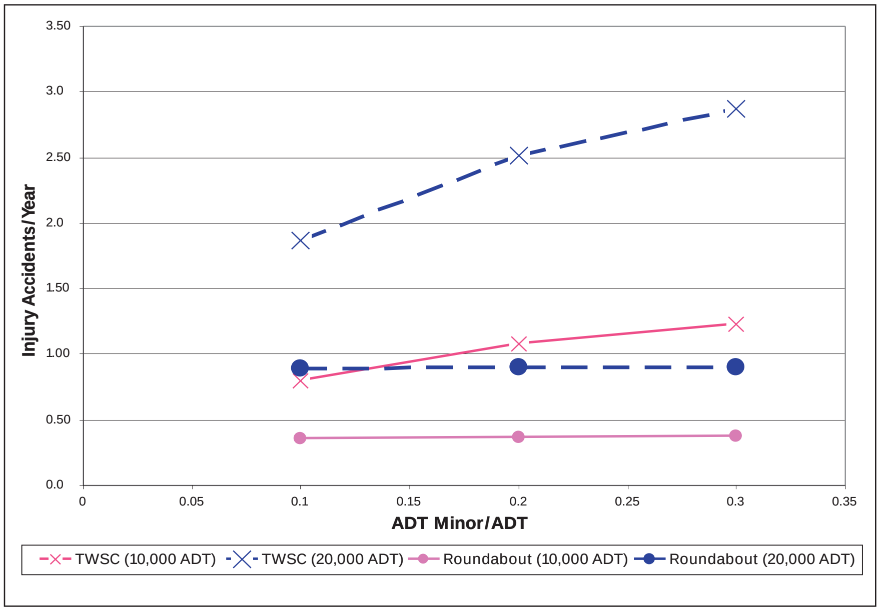
- Site visibility problems that reduce the effectiveness of stop sign control (in this case, landscaping of the roundabout needs to be carefully considered); and
- Inadequate separation of movements, especially on single-lane approaches.

Chapter 5 should be consulted for a more detailed analysis of the safety characteristics of roundabouts. There are currently a small number of roundabouts and therefore a relatively small crash record data base in the U.S. Therefore, it has not been possible to develop a national crash model for this intersection type. Roundabout crash prediction models have been developed for the United Kingdom (3). Crash models for conventional intersections in the United States are available (4, 5). Although crash data reporting may not be consistent between the U.K. and the U.S., comparison is plausible. The two sets of models have a key common measure of effectiveness in terms of injury and fatal crash frequency.

Therefore, for illustrative purposes, Exhibit 3-5 provides the results of injury crash prediction models for various ADT volumes of roundabouts versus rural TWSC intersections (6). The comparison shown is for a single-lane approach, four-leg roundabout with single-lane entries, and good geometric design. For the TWSC rural intersection model, the selected variables include rolling terrain, the main road as major collector, and a design speed of 80 km/h (50 mph). Rural roundabouts may experience approximately 66 percent fewer injury crashes than rural TWSC intersections for 10,000 entering ADT, and approximately 64 percent fewer crashes for 20,000 ADT. At urban roundabouts, the reduction will probably be smaller.

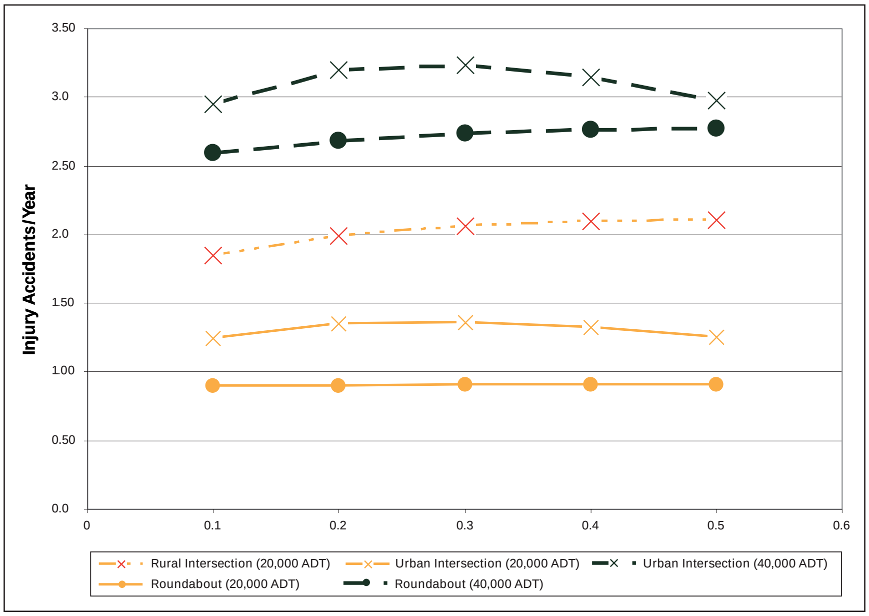
Also for illustration, Exhibit 3-6 provides the results of injury crash prediction models for various average daily traffic volumes at roundabouts versus rural and urban signalized intersections (6). The selected variables of the crash model for signalized (urban/suburban) intersections include multiphase fully-actuated signal, with a speed of 80 km/h (50 mph) on the major road. The 20,000 entering ADT is applied to single-lane roundabout approaches with four-legs. The 40,000 ADT is applied to double-lane roundabout approaches without flaring of the roundabout entries. In comparison to signalized intersections, roundabouts may experience approximately



Source: (6)

Exhibit 3-5. Comparison of predicted roundabout injury crashes with rural TWSC intersections.

Roundabouts have fewer annual injury crashes than rural two-way stop-controlled intersections, and the total number of crashes at roundabouts is relatively insensitive to minor street demand volumes.



Source: (6)

Exhibit 3-6. Comparison of predicted injury crashes for single-lane and double-lane roundabouts with rural or urban signalized intersections.

Roundabouts have fewer injury accidents per year than signalized intersections, particularly in rural areas. At volumes greater than 50,000 ADT, urban roundabout safety may be comparable to that of urban signalized intersections.

33 percent fewer injury crashes in urban and suburban areas and 56 percent fewer crashes in rural areas for 20,000 entering ADT. For 40,000 entering ADT, this reduction may only be about 15 percent in urban areas. Therefore, it is likely that roundabout safety may be comparable to signalized intersections at higher ADT (greater than 50,000).

These model comparisons are an estimation of mean crash frequency or average safety performance from a random sample of four-leg intersections from different countries and should be supplemented by engineering judgment and attention to safe design for all road users.

3.4.4 Operational improvement

A roundabout may be considered as a logical choice if its estimated performance is better than alternative control modes, usually either stop or signal control. The performance evaluation models presented in the next chapter provide a sound basis for comparison, but their application may require more effort and resources than an agency is prepared to devote in the planning stage. To simplify the selection process, the following assumptions are proposed for a planning-level comparison of control modes:

General delay and capacity comparisons between roundabouts and other forms of intersection control.

1. A roundabout will always provide a higher capacity and lower delays than AWSC operating with the same traffic volumes and right-of-way limitations.
2. A roundabout is unlikely to offer better performance in terms of lower overall delays than TWSC at intersections with minor movements (including cross street entry and major street left turns) that are not experiencing, nor predicted to experience, operational problems under TWSC.
3. A single-lane roundabout may be assumed to operate within its capacity at any intersection that does not exceed the peak-hour volume warrant for signals.
4. A roundabout that operates within its capacity will generally produce lower delays than a signalized intersection operating with the same traffic volumes and right-of-way limitations.

The above assumptions are documented in the literature (7) or explained by the analyses in Section 3.5. Collectively, they provide a good starting point for further analysis using procedures in Chapter 4. Although a roundabout may be the optimal control type from a vehicular operation standpoint, the relative performance of this control alternative for other modes should also be taken into consideration, as explained in Chapter 4.

3.4.4.1 Roundabout performance at flow thresholds for peak hour signal warrants

There are no warrants for roundabouts included in the *Manual of Uniform Traffic Control Devices* (MUTCD) (8), and it may be that roundabouts are not amenable to a warranting procedure. In other words, each roundabout should be justified on its own merits as the most appropriate intersection treatment alternative. It is, however, useful to consider the case in which the traffic volumes just meet the MUTCD warrant thresholds for traffic signals. For purposes of this discussion, the MUTCD peak hour warrant will be applied with a peak hour factor (PHF) of 0.9. Thus, the evaluation will reflect the performance in the heaviest 15 minutes of the peak hour.

Roundabout delays were compared with the corresponding values for TWSC, AWSC, and signals. A single-lane roundabout was assumed because the capacity of a single lane roundabout was adequate for all cases at the MUTCD volume warrant thresholds. SIDRA analysis software was used to estimate the delay for the various control alternatives because SIDRA was the only program readily available at the time this guide was developed that modeled all of the control alternatives (9).

The MUTCD warrant thresholds are given in terms of the heaviest minor street volume and sum of the major street volumes. Individual movement volumes may be obtained from the thresholds by assuming a directional factor, D, and left turn proportions. A “D” factor of 0.58 was applied to this example. Left turns on all approaches were assumed to be 10 to 50 percent of the total approach volume. In determining the MUTCD threshold volumes, two lanes were assumed on the major street and one lane on the minor street.

Based on these assumptions, the average delays per vehicle for signals and roundabouts are presented in Exhibit 3-7. These values represent the approach delay as perceived by the motorist. They do not include the geometric delay incurred within the roundabout. It is clear from this figure that roundabout control delays are substantially lower than signal delays, but in neither case are the delays excessive.

Similar comparisons are not presented for TWSC, because the capacity for minor street vehicles entering the major street was exceeded in all cases at the signal

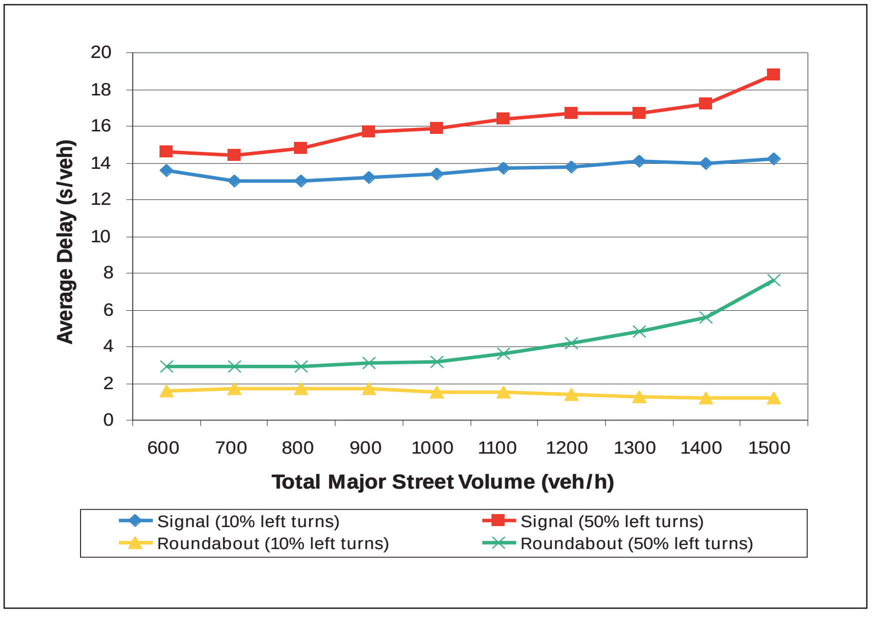


Exhibit 3-7. Average delay per vehicle at the MUTCD peak hour signal warrant threshold (excluding geometric delay).

Roundabout approach delay is relatively insensitive to total major street volume, but is sensitive to the left-turn percentage.

warrant thresholds. AWSC was found to be feasible only under a limited range of conditions: a maximum of 20 percent left turns can be accommodated when the major street volume is low and only 10 percent can be accommodated when the major street volume is high. Note that the minor street volume decreases as the major street volume increases at the signal warrant threshold.

This analysis of alternative intersection performance at the MUTCD peak hour volume signal warrant thresholds indicates that the single-lane roundabout is very competitive with all other forms of intersection control.

3.4.5 Special situations

It is important that the selection process not discourage the construction of a roundabout at any location where a roundabout would be a logical choice. Some flexibility must be built into the process by recognizing that the selection categories above are not all-inclusive. There may still be other situations that suggest that a roundabout would be a sensible control choice. Many of these situations are associated with unusual alignment or geometry where other solutions are intractable.

3.5 Comparing Operational Performance of Alternative Intersection Types

If a roundabout is being considered for operational reasons, then it may be compared with other feasible intersection control alternatives such as TWSC, AWSC, or signal control. This section provides approximate comparisons suitable for planning.

3.5.1 Two-way stop-control alternative

The majority of intersections in the U.S. operate under TWSC, and most of those intersections operate with minimal delay. The installation of a roundabout at a TWSC intersection that is operating satisfactorily will be difficult to justify on the basis of performance improvement alone, and one of the previously described selection categories is likely to be more appropriate.

Roundabouts may offer an effective solution at TWSC intersections with heavy left turns from the major street.

The two most common problems at TWSC intersections are congestion on the minor street caused by a demand that exceeds capacity, and queues that form on the major street because of inadequate capacity for left turning vehicles yielding to opposing traffic. Roundabouts may offer an effective solution to traffic problems at TWSC intersections with heavy left turns from the major route because they provide more favorable treatment to left turns than other control modes. "T" intersections are especially good candidates in this category because they tend to have higher left turning volumes.

Roundabouts work better when the proportion of minor street traffic is higher.

On the other hand, the problems experienced by low-volume cross street traffic at TWSC intersections with heavy through volumes on the major street are very difficult to solve by any traffic control measure. Roundabouts are generally not the solution to this type of problem because they create a significant impediment to the major movements. This situation is typical of a residential street intersection with a major arterial. The solution in most cases is to encourage the residential traffic to enter the arterial at a collector road with an intersection designed to accommodate higher entering volumes. The proportion of traffic on the major street is an important consideration in the comparison of a roundabout with a conventional four-leg intersection operating under TWSC. High proportions of minor street traffic tend to favor roundabouts, while low proportions favor TWSC.

An example of this may be seen in Exhibit 3-8, which shows the AADT capacity for planning purposes as a function of the proportion of traffic on the major street. The assumptions in this exhibit are the same as those that have been described previously in Section 3.3. Constant proportions of 10 percent right turns (which were ignored in roundabout analysis) and 20 percent left turns were used for all movements. As expected, the roundabout offers a much higher capacity at lower proportions of major street traffic. When the major and minor street volumes are equal, the roundabout capacity is approximately double that of the TWSC intersection. It is interesting to note that the two capacity values converge at the point where the minor street proportion becomes negligible. This effect confirms the expectation that a roundabout will have approximately the same capacity as a stop-controlled intersection when there is no cross street traffic.

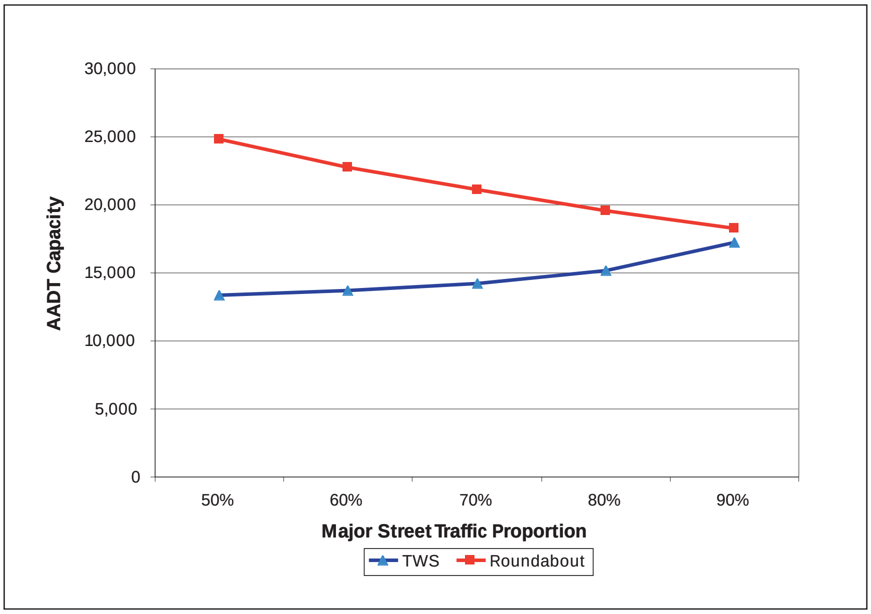


Exhibit 3-8. Comparison of TWSC and single-lane roundabout capacity.

Roundabout capacity decreases as the proportion of minor street entering traffic decreases. Roundabouts and TWSC intersections have about the same capacity when the minor street proportion is less than 10 percent.

3.5.2 All-way stop-control alternative

When cross street traffic volumes are heavy enough to meet the MUTCD warrants for AWSC control, roundabouts become an especially attractive solution because of their higher capacities and lower delays. The selection of a roundabout as an alternative to AWSC should emphasize cost and safety considerations, because roundabouts always offer better performance for vehicles than AWSC, given the same traffic conditions. Roundabouts that are proposed as alternatives to stop control would typically have single-lane approaches.

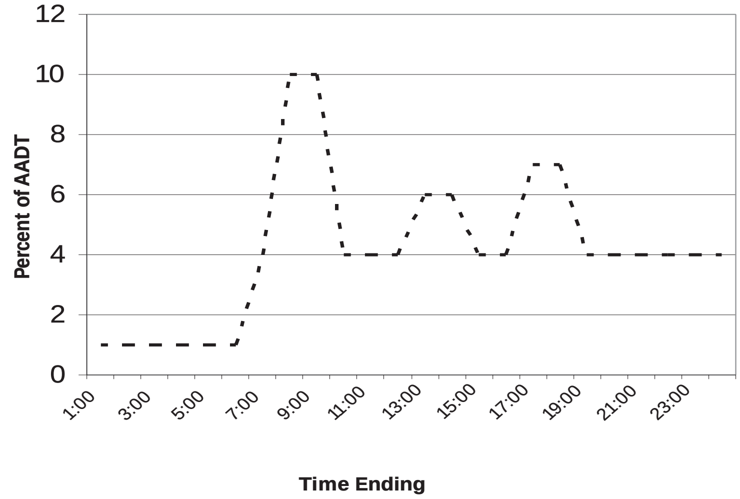
A substantial part of the benefit of a roundabout compared to an all-way stop intersection is obtained during the off-peak periods, because the restrictive stop control applies for the entire day. The MUTCD does not permit stop control on a part-time basis. The extent of the benefit will depend on the amount of traffic at the intersection and on the proportion of left turns. Left turns degrade the operation of all traffic control modes, but they have a smaller effect on roundabouts than on stop signs or signals.

A substantial part of the delay-reduction benefit of roundabouts, compared to AWSC intersections, comes during off-peak periods.

The planning level analysis that began earlier in this chapter may be extended to estimate the benefits of a roundabout compared to AWSC. Retaining the previous assumptions about the directional and temporal distribution factors for traffic volumes (i.e., K=0.1, D=0.58), it is possible to analyze both control modes throughout an entire 24-hour day. Only one additional set of assumptions is required. It is necessary to construct an assumed hourly distribution of traffic throughout the day that conforms to these two factors.

A reasonably typical sample distribution for this purpose is illustrated in Exhibit 3-9, which would generally represent inbound traffic to employment centers, because of the larger peak in the AM period, accompanied by smaller peaks in the noontime and PM periods. Daytime off-peak periods have 4 percent of the AADT per hour, and late-night off-peak periods (midnight to 6 AM) have 1 percent.

Exhibit 3-9. Sample hourly distribution of traffic.



The outbound direction may be added as a mirror image of the inbound direction, keeping the volumes the same as the inbound during the off-peak periods and applying the D factor of 0.58 during the AM and PM peaks. This distribution was used in the estimation of the benefits of a roundabout compared to the AWSC mode. It was also used later for comparison with traffic signal operations. For purposes of estimating annual delay savings, a total of 250 days per year is assumed. This provides a conservative estimate by eliminating weekends and holidays.

The comparisons were performed using traffic operations models that are described in Chapter 4 of this guide. The SIDRA model was used to analyze both the roundabout and AWSC operation, because SIDRA was the only model readily available at the time this guide was developed that treated both of these types of control. SIDRA provides an option to either include or omit the geometric delay experienced within the intersection. The geometric delay was included for purposes of estimating annual benefits. It was excluded in Section 3.4.4.1 that dealt with driver-perceived approach delay.

The results of this comparison are presented in Exhibit 3-10 and Exhibit 3-11 in terms of potential annual savings in delay of a single-lane roundabout over an AWSC intersection with one lane on all approaches, as a function of the proportion of left turning traffic for single-lane approaches for volume distributions of 50 percent and 65 percent on the major street, respectively. Each exhibit has lines representing 10 percent, 25 percent, and 33 percent left turn proportions.

Note that the potential annual benefit is in the range of 5,000 to 50,000 vehicle-hours per year. The benefit increases substantially with increasing AADT and left turn proportions. The comparison terminates in each case when the capacity of the AWSC operation is exceeded. No comparisons were made beyond 18,000 AADT, because AWSC operation is not practical beyond that level.

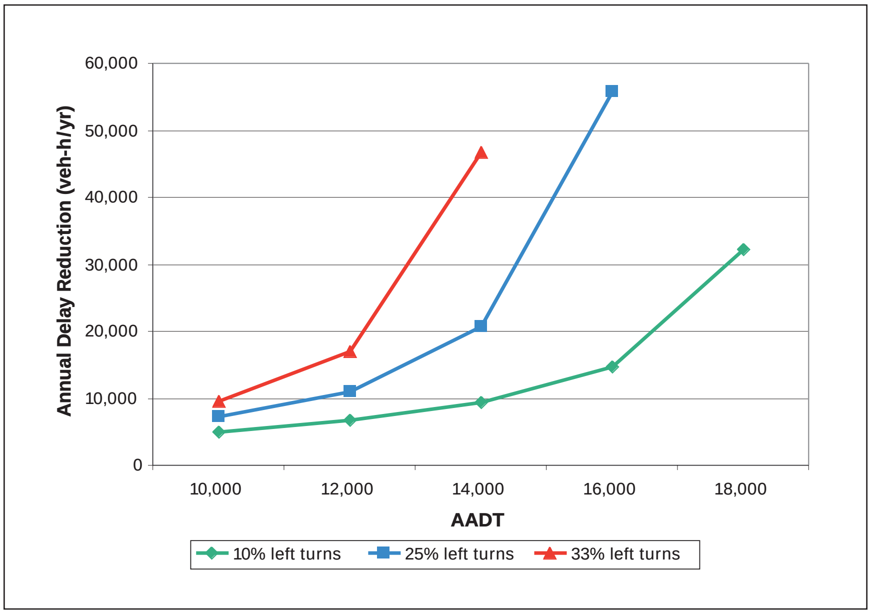


Exhibit 3-10. Annual savings in delay of single-lane roundabout versus AWSC, 50 percent of volume on the major street.

The delay-reduction benefit of roundabouts, compared to AWSC, increases as left-turn volumes, major street proportion, and AADT increase.

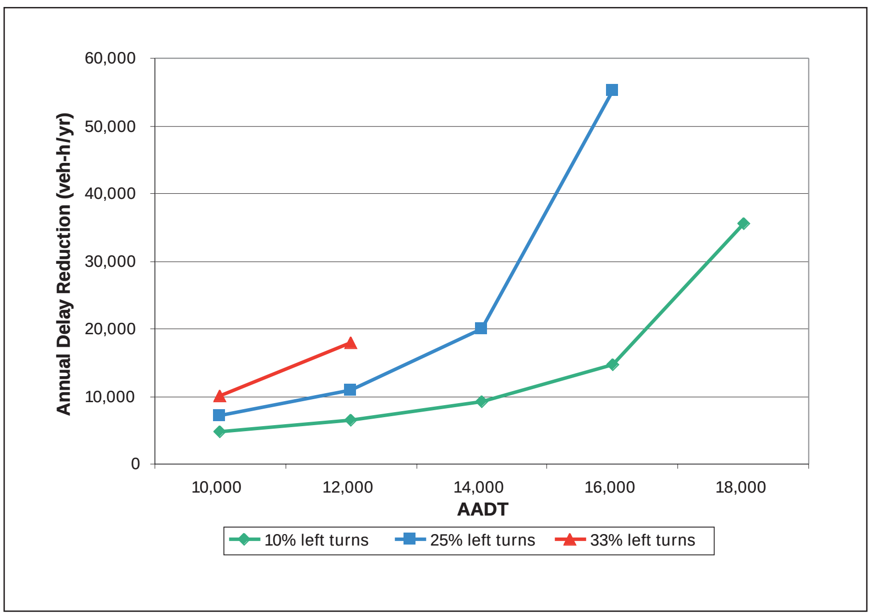


Exhibit 3-11. Annual savings in delay of single-lane roundabout versus AWSC, 65 percent of volume on the major street.

3.5.3 Signal control alternative

When traffic volumes are heavy enough to warrant signalization, the selection process becomes somewhat more rigorous. The usual basis for selection here is that a roundabout will provide better operational performance than a signal in terms of stops, delay, fuel consumption, and pollution emissions. For planning purposes, this may generally be assumed to be the case provided that the roundabout is operating within its capacity. The task then becomes to assess whether any roundabout configuration can be made to work satisfactorily. If not, then a signal or grade separation are remaining alternatives. As in the case of stop control, intersections with heavy left turns are especially good roundabout candidates.

The graphical approximation presented earlier for capacity estimation should be useful at this stage. The results should be considered purely as a planning level estimate, and it must be recognized that this estimate will probably change during the design phase. Users of this guide should also consult the most recent version of the *Highway Capacity Manual* (HCM) (10) as more U.S. data and consensus on modeling U.S. roundabout performance evolves.

As in the case of AWSC operations, some of the most important benefits of a roundabout compared to a traffic signal will accrue during the off-peak periods. The comparison of delay savings discussed previously has therefore been extended to deal with traffic signals as well as stop signs. The same temporal distribution of traffic volumes used for the roundabout-AWSC comparison was assumed.

The signal timing design was prepared for each of the conditions to accommodate traffic in the heaviest peak period. The traffic actuated controller was allowed to respond to fluctuations in demand during the rest of the day using its own logic. This strategy is consistent with common traffic engineering practice. All approaches were considered to be isolated and free of the influence of coordinated systems. Left turn protection was provided for the whole day for all approaches with a volume cross-product (i.e., the product of the left turn and opposing traffic volumes) of 60,000 or greater during the peak period. When left turn protection was provided, the left turns were also allowed to proceed on the solid green indication (i.e., protected-plus-permitted operation).

The results of this comparison are presented in Exhibit 3-12 for 50 percent major street traffic and Exhibit 3-13 for 65 percent major street traffic. Both cases include AADT values up to 34,000 vehicles per day. Single-lane approaches were used for both signals and roundabouts with AADTs below 25,000 vehicles per day. Two-lane approaches were assumed beyond that point. All signalized approaches were assumed to have left turn bays.

Benefits may continue to accrue beyond the 34,000 AADT level but the design parameters for both the signal and the roundabout are much more difficult to generalize for planning level analyses. When AADTs exceed 34,000 vehicles per day, performance evaluation should be carried out using the more detailed procedures presented in Chapter 4 of this guide.

The selection of a roundabout as an alternative to signal control will be much simpler if a single-lane roundabout is estimated to have adequate capacity. If, on the other hand, it is determined that one or more legs will require more than one entry lane, some preliminary design work beyond the normal planning level will generally be required to develop the roundabout configuration and determine the space requirements.

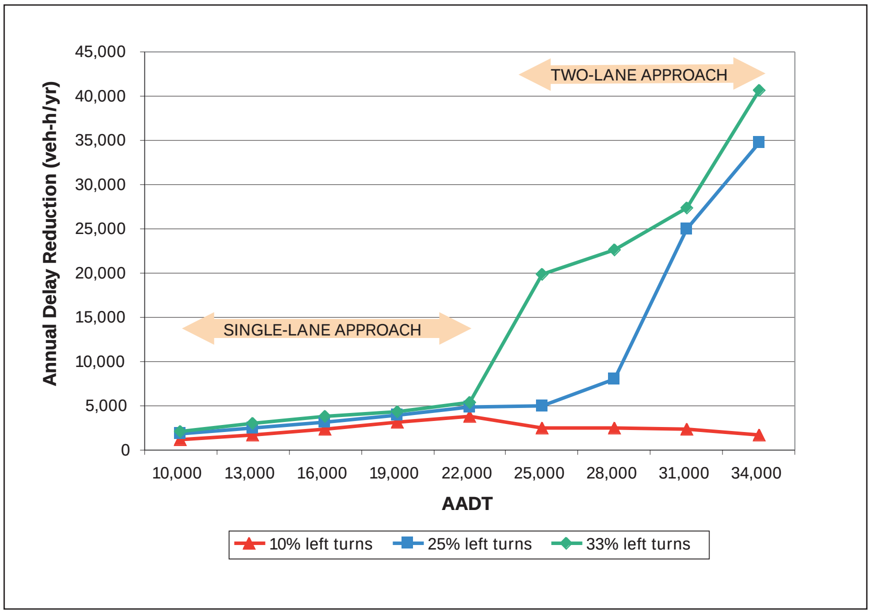


Exhibit 3-12. Delay savings for roundabout vs. signal, 50 percent volume on major street.

When volumes are evenly split between major and minor approaches, the delay savings of roundabouts versus signals are especially notable on two-lane approaches with high left turn proportions.

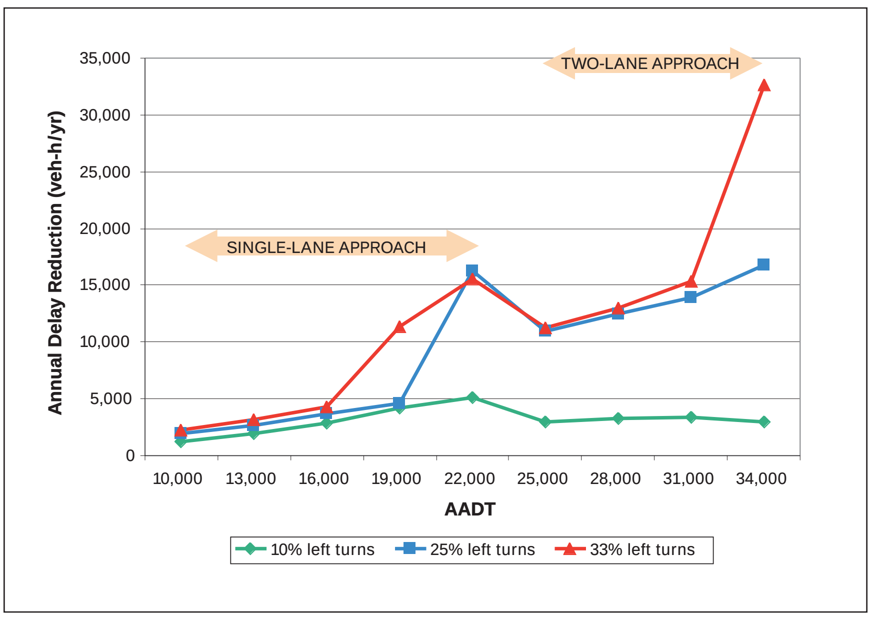


Exhibit 3-13. Delay savings for roundabout vs. signal, 65 percent volume on major street.

When the major street approaches dominate, roundabout delay is lower than signal delay, particularly at the upper volume limit for single-lane approaches and when there is a high proportion of left turns.

3.6 Space Requirements

Roundabouts that are designed to accommodate vehicles larger than passenger cars or small trucks typically require more space than conventional intersections. However, this may be more than offset by the space saved compared with turning lane requirements at alternative intersection forms. The key indicator of the required space is the inscribed circle diameter. A detailed design is required to determine the space requirements at a specific site, especially if more than one lane is needed to accommodate the entering and circulating traffic. This is, however, another case in which the use of assumptions and approximations can produce

The design templates in Appendix B may be used to determine initial space requirements for the appropriate roundabout category.

preliminary values that are adequate for planning purposes. For initial space requirements, the design templates in Appendix B for the most appropriate of the six roundabout categories for the specific site may be consulted.

One important question is whether or not the proposed roundabout will fit within the existing property lines, or whether additional right-of-way will be required. Four examples have been created to demonstrate the spatial effects of comparable intersection types, and the assumptions are summarized in Exhibit 3-14. Note that there are many combinations of turning volumes that would affect the actual lane configurations and design storage lengths. Therefore, these examples should not be used out of context.

Exhibit 3-14. Assumptions for spatial comparison of roundabouts and comparable conventional intersections.

	Roundabout Type		Conventional Intersection	
Category	Main Street Approach Lanes	Side Street Approach Lanes	Main Street Approach Lanes	Side Street Approach Lanes
Urban compact	1	1	1	1
Urban single-lane	1	1	1 + LT pocket	1
Urban double-lane	2	1	2 + LT pocket	1 + LT pocket
Urban double-lane with flaring	1 flared to 2	1	2 + LT pocket	1 + LT pocket

Note: LT = left turn

Although roundabouts typically require more area at the junction compared to conventional intersections, they may not need as much area on the approaches.

As can be seen in Exhibit 3-15 through Exhibit 3-18, roundabouts typically require more area at the junction than conventional intersections. However, as capacity needs increase the size of the roundabout and comparable conventional (signalized) intersection, the increase in space requirements are increasingly offset by a reduction in space requirements on the approaches. This is because the widening or flaring required for a roundabout can be accomplished in a shorter distance than is typically required to develop left turn lanes and transition tapers at conventional intersections.

As can be seen in Exhibit 3-18, flared roundabouts offer the most potential for reducing spatial requirements on the approaches as compared to conventional intersections. This effect of providing capacity at the intersections while reducing lane requirements between intersections, known as "wide nodes and narrow roads," is discussed further in Chapter 8.

3.7 Economic Evaluation

Economic evaluation is an important part of any public works planning process. For roundabout applications, economic evaluation becomes important when compar-

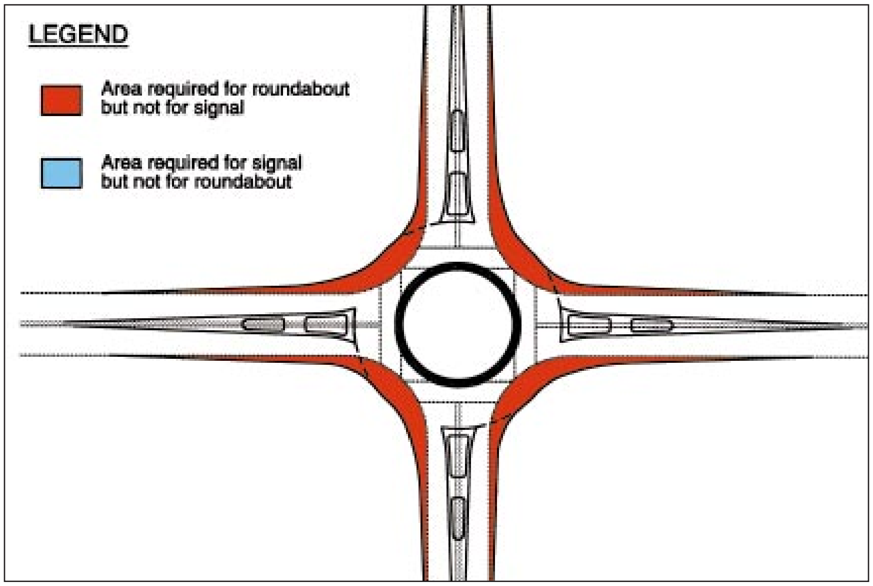


Exhibit 3-15. Area comparison: Urban compact roundabout vs. comparable signalized intersection.

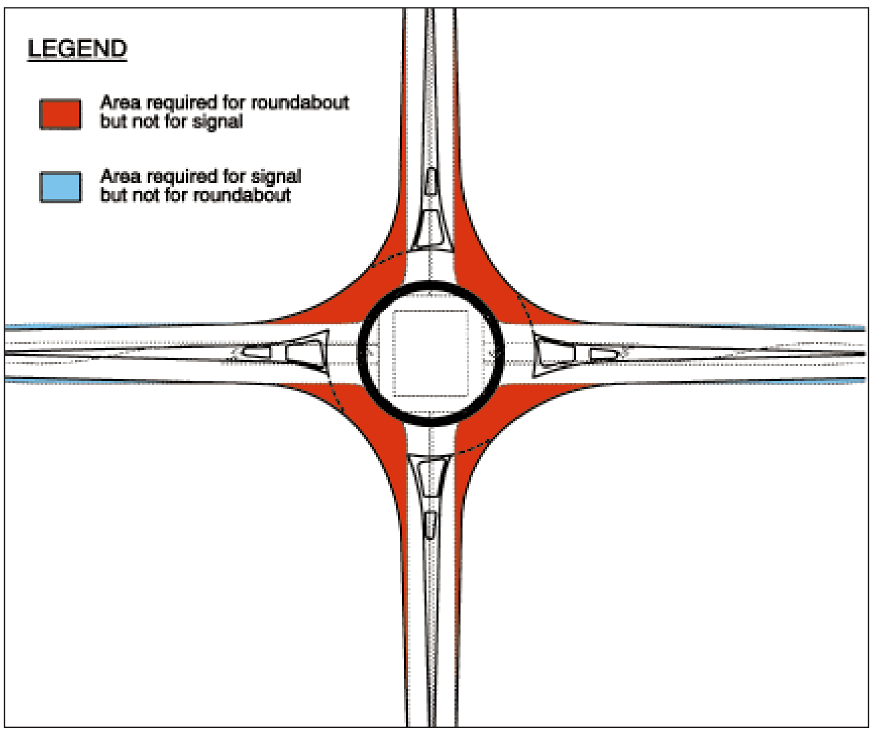


Exhibit 3-16. Area comparison: Urban single-lane roundabout vs. comparable signalized intersection.

Exhibit 3-17. Area comparison: Urban double-lane roundabout vs. comparable signalized intersection.

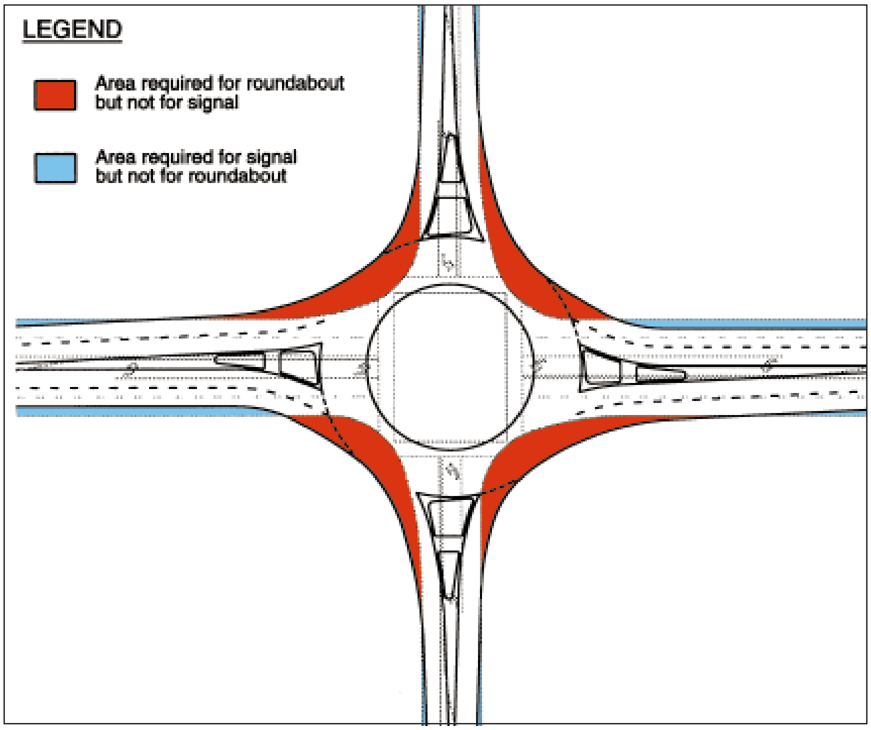
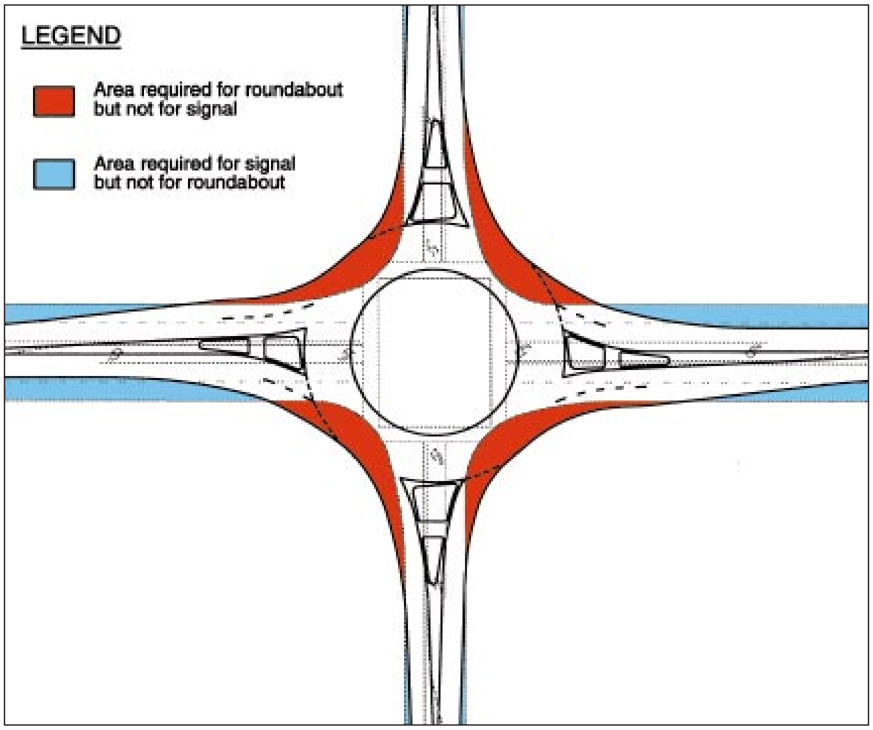


Exhibit 3-18. Area comparison: Urban flared roundabout vs. comparable signalized intersection.



Urban flared roundabouts in particular illustrate the "wide nodes, narrow roads" concept discussed further in Chapter 8.

ing roundabouts against other forms of intersections and traffic control, such as comparing a roundabout with a signalized intersection.

The most appropriate method for evaluating public works projects of this type is usually the benefit-cost analysis method. The following sections discuss this method as it typically applies to roundabout evaluation, although it can be generalized for most transportation projects.

3.7.1 Methodology

The benefit-cost method is elaborated on in detail in a number of standard references, including the ITE *Transportation Planning Handbook* (11) and various American Association of State Highway and Transportation Officials (AASHTO) publications (12, 13). The basic premise of this method of evaluation is to compare the incremental benefit between two alternatives to the incremental costs between the same alternatives. Assuming Alternatives A and B, the equation for calculating the incremental benefit-cost ratio of Alternative B relative to Alternative A is given in Equation 3-1.

$$B/C_{B\,A} = \frac{Benefits_B - Benefits_A}{Costs_B - Costs_A} \tag{3-1}$$

Benefit-cost analysis typically takes two forms. For assessing the viability of a number of alternatives, each alternative is compared individually with a no-build alternative. If the analysis for Alternative A relative to the no-build alternative indicates a benefit-cost ratio exceeding 1.0, Alternative A has benefits that exceed its costs and is thus a viable project.

For ranking alternatives, the incremental benefit-cost ratio analysis is used to compare the relative benefits and costs between alternatives. Projects should not be ranked based on their benefit-cost ratio relative to the no-build alternative. After eliminating any alternatives that are not viable as compared to the no-build alternative, alternatives are compared in a pair-wise fashion to establish the priority between projects.

Rank alternatives based on their incremental benefit-cost ratio, not on their ratio relative to the no-build alternative.

Since many of the input parameters may be estimated, a rigorous analysis should consider varying the parameter values of key assumptions to verify that the recommended alternative is robust, even under slightly varying assumptions, and under what circumstances it may no longer be preferred.

3.7.2 Estimating benefits

Benefits for a public works project are generally comprised of three elements: safety benefits, operational benefits, and environmental benefits. Each benefit is typically quantified on an annualized basis and so is readily usable in a benefit-cost analysis. The following sections discuss these in more detail.

Benefits consist of:

- **Safety benefits**
- **Operational benefits**
- **Environmental benefits**

3.7.2.1 Safety benefits

Safety benefits are defined as the assumed savings to the public due to a reduction in crashes within the project area. The general procedure for determining safety benefits is as follows:

- Quantify the existing safety history in the study area in terms of a crash rate for each level of severity (fatal, injury, property damage). This rate, expressed in terms of crashes per million entering vehicles, is computed by dividing the number of crashes of a given severity that occurred during the "before" period by the number of vehicles that entered the intersection during the same period. This results in a "before" crash rate for each level of severity.
- Estimate the change in crashes of each level of severity that can be reasonably expected due to the proposed improvements. As documented elsewhere in this guide, roundabouts tend to have proportionately greater reductions in fatal and injury crashes than property damage crashes.
- Determine a new expected crash rate (an "after" crash rate) by multiplying the "before" crash rates by the expected reductions. It is best to use local data to determine appropriate crash reduction factors due to geometric or traffic control changes, as well as the assumed costs of various severity levels of crashes.
- Estimate the number of "after" crashes of each level of severity for the life of the project by multiplying the "after" crash rate by the expected number of entering vehicles over the life of the project.
- Estimate a safety benefit by multiplying the expected number of "after" crashes of each level of severity by the average cost of each crash and then annualizing the result. The values in Exhibit 3-19 can provide a starting point, although local data should be used where available.

Exhibit 3-19. Estimated costs for crashes of varying levels of severity.

Crash Severity	Economic Cost (1997 dollars)
Death (per death)	$980,000
Injury (per injury)	$34,100
Property Damage Only (per crash)	$6,400

Source: National Safety Council (14)

3.7.2.2 Operational benefits

Quantify operational benefits in terms of vehicle-hours of delay.

The operational benefits of a project may be quantified in terms of the overall reduction in person-hours of delay to the public. Delay has a cost to the public in terms of lost productivity, and thus a value of time can typically be assigned to changes in estimated delay to quantify benefits associated with delay reduction.

The calculation of annual person-hours of delay can be performed with varying levels of detail, depending on the availability of data. For example, the vehicle-hours of delay may be computed as follows. The results should be converted to person-hours of delay using appropriate vehicle-occupancy factors (including transit), then adding pedestrian delay if significant.

- Estimate the delay per vehicle for each hour of the day. If turning movements are available for multiple hours, this estimate can be computed directly. If only the peak hour is available, the delay for an off-peak hour can be approximated by proportioning the peak hour turning movements by total entering vehicles.
- Determine the daily vehicle-hours of delay by multiplying the estimated delay per vehicle for a given hour by the total entering vehicles during that hour and then aggregating the results over the entire day. If data is available, these calculations can be separated by day of week or by weekday, Saturday, and Sunday.
- Determine annual vehicle-hours of delay by multiplying the daily vehicle-hours of delay by 365. If separate values have been calculated by day of week, first determine the weekday vehicle-hours of delay and then multiply by 52.1 (365 divided by 7). It may be appropriate to use fewer than 365 days per year because the operational benefits will not usually apply equally on all days.

3.7.2.3 Environmental benefits

The environmental benefits of a project are most readily quantified in terms of reduced fuel consumption and improved air quality. Of these, reductions in fuel consumption and the benefits associated with those reductions are typically the simplest to determine.

One way to determine fuel consumption is to use the same procedure for estimating delay, as described previously. Fuel consumption is an output of several of the models in use today, although the user is cautioned to ensure that the model is appropriately calibrated for current U.S. conditions. Alternatively, one can estimate fuel consumption by using the estimate of annual vehicle-hours of delay and then multiplying that by an assumed fuel consumption rate during idling, expressed as liters per hour (gallons per hour) of idling. The resulting estimate can then be converted to a cost by assuming an average cost of fuel, expressed in dollars per liter (dollars per gallon).

3.7.3 Estimation of costs

Costs for a public works project are generally comprised of two elements: capitalized construction costs and operations and maintenance (O&M) costs. Although O&M costs are typically determined on an annualized basis, construction costs are typically a near-term activity that must be annualized. The following sections discuss these in more detail.

3.7.3.1 Construction costs

Construction costs for each alternative should be calculated using normal preliminary engineering cost estimating techniques. These costs should include the costs of any necessary earthwork, paving, bridges and retaining walls, signing and striping, illumination, and signalization.

To convert construction costs into an annualized value for use in the benefit-cost analysis, a *capital recovery factor* (CRF) should be used, shown in Equation 3-2. This converts a present value cost into an annualized cost over a period of *n* years using an assumed discount rate of *i* percent.

$$CRF = \frac{i(1+i)^n}{i(1+i)^n - 1} \quad (3\text{-}2)$$

where: i = discount rate
n = number of periods (years)

3.7.3.2 Operation and maintenance (O&M) costs

Roundabout O&M costs are typically slightly higher than signalized intersections for:
- **Illumination**
- **Signing**
- **Pavement marking**
- **Landscaping**

Operation and maintenance costs vary significantly between roundabouts and other forms of intersection control beyond the basic elements. Common elements include signing and pavement marking maintenance and power for illumination, if provided.

Roundabouts typically have a slightly higher illumination power and maintenance costs compared to signalized or sign-controlled intersections due to a larger number of illumination poles. Roundabouts have slightly higher signing and pavement marking maintenance costs due to a higher number of signs and pavement markings. Roundabouts also introduce additional cost associated with the maintenance of any landscaping in and around the roundabout.

Signalized intersections also have O&M costs for:
- **Signal power**
- **Bulb replacement**
- **Detection maintenance**

Signalized intersections have considerable additional cost associated with power for the traffic signal and maintenance costs such as bulb replacement, detection maintenance, etc. Power costs vary considerably from region to region and over time and should be verified locally. For general purposes, an annual cost of $3,000 for providing power to a signalized intersection is a reasonable approximation.

3.8 References

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4 Operation

Chapter 4 Operation

This chapter presents methods for analyzing the operation of an existing or planned roundabout. The methods allow a transportation analyst to assess the operational performance of a facility, given information about the usage of the facility and its geometric design elements. An operational analysis produces two kinds of estimates: (1) the capacity of a facility, i.e., the ability of the facility to accommodate various streams of users, and (2) the level of performance, often measured in terms of one or more measures of effectiveness, such as delay and queues.

The *Highway Capacity Manual* (1) (HCM) defines the *capacity* of a facility as "the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic, and control conditions." While capacity is a specific measure that can be defined and estimated, *level of service* (LOS) is a qualitative measure that "characterizes operational conditions within a traffic stream and their perception by motorists and passengers." To quantify level of service, the HCM defines specific *measures of effectiveness* for each highway facility type. *Control delay* is the measure of effectiveness that is used to define level of service at intersections, as perceived by users. In addition to control delay, all intersections cause some drivers to also incur *geometric delays* when making turns. A systems analysis of a roadway network may include geometric delay because of the slower vehicle paths required for turning through intersections. An example speed profile is shown in Chapter 6 to demonstrate the speed reduction that results from geometric delay at a roundabout.

Roundabouts produce both control delay and geometric delay.

While an operational analysis can be used to evaluate the performance of an existing roundabout during a base or future year, its more common function in the U.S. may be to evaluate new roundabout designs.

This chapter:

- Describes traffic operations at roundabouts;
- Lists the data required to evaluate the performance of a roundabout;
- Presents a method to estimate the capacity of five of the six basic roundabout configurations presented in this guide;
- Describes the measures of effectiveness used to determine the performance of a roundabout and a method to estimate these measures; and
- Briefly describes the computer software packages available to implement the capacity and performance analysis procedures.

Appendix A provides background information on the various capacity relationships.

4.1 Traffic Operation at Roundabouts

4.1.1 Driver behavior and geometric elements

A roundabout brings together conflicting traffic streams, allows the streams to safely merge and traverse the roundabout, and exit the streams to their desired directions. The geometric elements of the roundabout provide guidance to drivers approaching, entering, and traveling through a roundabout.

Approach speed is governed by:
- **Approach roadway width**
- **Roadway curvature**
- **Approach volume**

Drivers approaching a roundabout must slow to a speed that will allow them to safely interact with other users of the roundabout, and to negotiate the roundabout. The width of the approach roadway, the curvature of the roadway, and the volume of traffic present on the approach govern this speed. As drivers approach the yield line, they must check for conflicting vehicles already on the circulating roadway and determine when it is safe and prudent to enter the circulating stream. The widths of the approach roadway and entry determine the number of vehicle streams that may form side by side at the yield line and govern the rate at which vehicles may enter the circulating roadway. The size of the inscribed circle affects the radius of the driver's path, which in turn determines the speed at which drivers travel on the roundabout. The width of the circulatory roadway determines the number of vehicles that may travel side by side on the roundabout.

The British (2), French (3), and German (4) analytical procedures are based on empirical relationships that directly relate capacity to both traffic characteristics and roundabout geometry. The British empirical relationships reveal that small sublane changes in the geometric parameters produce significant changes in capacity.

For instance, if some approaches are flared or have additional short lanes, these provide considerably more capacity for two reasons. First, wider entries require wider circulatory roadway widths. This provides for more opportunities for the circulatory traffic to bunch together, thus increasing the number of acceptable opportunities to enter, thereby increasing capacity. Second, the typical size of groups of drivers entering into acceptable opportunities in the circulatory traffic is quite small, so short lanes can be very effective in increasing group sizes, because the short lane is frequently able to be filled.

The British (2) use the inscribed circle diameter, the entry width, the approach (road) half width, the entry radius, and the sharpness of the flare to define the performance of a roundabout. The sharpness of the flare, *S*, is a measure of the rate at which the extra width is developed in the entry flare. Large values of *S* correspond to short, severe flares, and small values of *S* correspond to long, gradual flares (5).

Geometric elements that affect entry capacity include:
- **Approach half width**
- **Entry width**
- **Entry angle**
- **Average effective flare length**

The results of the extensive empirical British research indicate that approach half width, entry width, average effective flare length and entry angle have the most significant effect on entry capacity. Roundabouts fit into two general classes: those with a small inscribed circle diameter of less than 50 m (165 ft.) and those with a diameter above 50 m. The British relationships provide a means of including both of these roundabout types. The inscribed circle diameter has a relatively small effect for inscribed diameters of 50 m (165 ft) or less. The entry radius has little effect on capacity provided that it is 20 m (65 ft) or more. The use of perpendicular entries (70 degrees or more) and small entry radii (less than 15 m [50 ft]) will reduce capacity. The presence of the geometric parameters in the British and French models allow designers to manipulate elements of their design to determine both their operational and safety effects. German research has not been able to find the same influence of geometry, although this may be due to the relatively narrow range of geometries in Germany (4).

Perpendicular entries and small entry radii reduce capacity; inscribed circle diameters of 50 m (165 ft) or less have little effect on capacity.

Thus, the geometric elements of a roundabout, together with the volume of traffic desiring to use a roundabout at a given time, may determine the efficiency with which a roundabout operates.

4.1.2 Concept of roundabout capacity

The capacity of each entry to a roundabout is the maximum rate at which vehicles can reasonably be expected to enter the roundabout from an approach during a given time period under prevailing traffic and roadway (geometric) conditions. An operational analysis considers a precise set of geometric conditions and traffic flow rates defined for a 15-minute analysis period for each roundabout entry. While consideration of Average Annual Daily Traffic volumes (AADT) across all approaches is useful for planning purposes as provided in Exhibit 1-13 and Chapter 3, analysis of this shorter time period is critical to assessing the level of performance of the roundabout and its individual components.

Roundabout capacity defined.

Operational analyses consider 15-minute volumes, as opposed to the daily volumes used in planning analyses.

The capacity of the entire roundabout is not considered, as it depends on many terms. However, Exhibit 1-13 provides threshold average daily traffic volumes for the various categories of roundabouts, assuming four legs. Below these thresholds, a four-legged roundabout with roadways intersecting perpendicularly should have adequate capacity (provided the traffic volumes are reasonably balanced and the geometry does not deviate substantially from those shown on the design templates in Exhibits 1-7 through 1-12). The focus in this chapter on the roundabout entry is similar to the operational analysis methods used for other forms of unsignalized intersections and for signalized intersections. In each case, the capacity of the entry or approach is computed as a function of traffic on the other (conflicting) approaches, the interaction of these traffic streams, and the intersection geometry.

For a properly designed roundabout, the yield line is the relevant point for capacity analysis. The approach capacity is the capacity provided at the yield line. This is determined by a number of geometric parameters in addition to the entry width. On multilane roundabouts it is important to balance the use of each lane, because otherwise some lanes may be overloaded while others are underused. Poorly designed exits may influence driver behavior and cause lane imbalance and congestion at the opposite leg.

The approach capacity is the capacity provided at the yield line.

4.2 Data Requirements

The analysis method described in this chapter requires the specification of traffic volumes for each approach to the roundabout, including the flow rate for each directional movement. Volumes are typically expressed in passenger car vehicles per hour (vph), for a specified 15-minute analysis period. To convert other vehicle types to *passenger car equivalents* (pce), use the conversion factors given in Exhibit 4-1.

Different size vehicles have different capacity impacts; passenger cars are used as the basis for comparison.

Exhibit 4-1. Conversion factors for passenger car equivalents (pce).

Vehicle Type	Passenger Car Equivalent (pce)
Car	1.0
Single-unit truck or bus	1.5
Truck with trailer	2.0
Bicycle or motorcycle	0.5

Source: (6), (7)

Traffic volume data for an urban roundabout should be collected for each directional movement for at least the morning and evening peak periods, since the various movements, and thus approach and circulating volumes, may peak at different times. At rural roundabouts, the analyst should check the requirements of the agency with the jurisdiction of the site. The reader is referred to the *Manual of Transportation Engineering Studies* (8) for a complete discussion of traffic volume data collection methods. Typically, intersection volume counts are made at the intersection stop bar, with an observer noting the number of cars that pass that point over a specified time period. However, particularly with respect to cases in which demand exceeds capacity (when queues do not dissipate within the analysis period), it is important to note that the stop bar counts reflect only the volume that is served, not the demand volume. In this case, care must be taken to collect data upstream of the end of a queue so that true demand volumes are available for analysis.

Entry flow and circulating flow for each approach are the volumes of interest for roundabout capacity analysis, rather than turning movement volumes.

The relationship between the standard origin-to-destination turning movements at an intersection and the circulating and entry flows at a roundabout is important, yet is often complicated to compute, particularly if an intersection has more than four approaches. For conventional intersctions, traffic flow data are accumulated by directional turning movement, such as for the northbound left turn. For roundabouts, however, the data of interest for each approach are the entry flow and the circulating flow. Entry flow is simply the sum of the through, left, and right turn movements on an approach. Circulating flow is the sum of the vehicles from different movements passing in front of the adjacent uptstream splitter island. At existing roundabouts, these flows can simply be measured in the field. Right turns are included in approach volumes and require capacity, but are not included in the circulating volumes downstream because they exit before the next entrance.

For proposed or planned four-legged roundabouts, Equations 4-1 through 4-4 can be applied to determine conflicting (circulating) flow rates, as shown graphically in Exhibit 4-2.

Determining circulating volumes as a function of turning movement volumes.

$$V_{EB,circ} = V_{WB,LT} + V_{SB,LT} + V_{SB,TH} + V_{NB,U\text{-}turn} + V_{WB,U\text{-}turn} + V_{SB,U\text{-}turn} \quad (4\text{-}1)$$

$$V_{WB,circ} = V_{EB,LT} + V_{NB,LT} + V_{NB,TH} + V_{SB,U\text{-}turn} + V_{EB,U\text{-}turn} + V_{NB,U\text{-}turn} \quad (4\text{-}2)$$

$$V_{NB,circ} = V_{EB,LT} + V_{EB,TH} + V_{SB,LT} + V_{WB,U\text{-}turn} + V_{SB,U\text{-}turn} + V_{EB,U\text{-}turn} \quad (4\text{-}3)$$

$$V_{SB,circ} = V_{WB,LT} + V_{WB,TH} + V_{NB,LT} + V_{EB,U\text{-}turn} + V_{NB,U\text{-}turn} + V_{WB,U\text{-}turn} \quad (4\text{-}4)$$

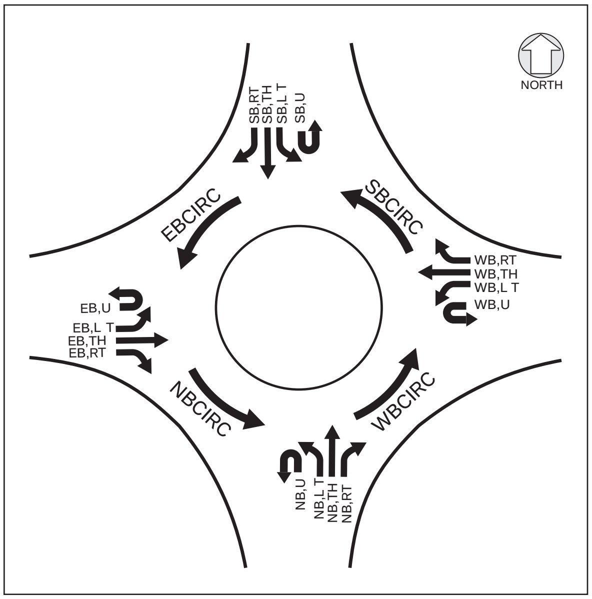


Exhibit 4-2. Traffic flow parameters.

For existing roundabouts, when approach, right-turn, circulating, and exit flows are counted, directional turning movements can be computed as shown in the following example. Equation 4-5 shows the through movement flow rate for the eastbound approach as a function of the entry flow rate for that approach, the exit flow rate for the opposing approach, the right turn flow rate for the subject approach, the right turn flow rate for the approach on the right, and the circulating flow rate for the approach on the right. Other through movement flow rates can be estimated using a similar relationship.

$$V_{EB,TH} = V_{EB,entry} + V_{WB,exit} - V_{EB,RT} - V_{NB,RT} - V_{NB,circ} \quad (4\text{-}5)$$

The left turn flow rate for an approach is a function of the entry flow rate, the through flow rate, and the right turn flow rate for that same approach, as shown in Equation 4-6. Again, other movements' flows are estimated using similar equations.

$$V_{EB,LT} = V_{EB,entry} - V_{EB,TH} - V_{EB,RT} \quad (4\text{-}6)$$

While this method is mathematically correct, it is somewhat sensitive to errors and inconsistencies in the input data. It is important that the counts at all of the locations in the roundabout be made simultaneously. Inconsistencies in the data from counts taken on different days can produce meaningless results, including negative volumes. At a minimum, the sum of the entering and exiting volumes should be checked and adjustments should be made if necessary to ensure that the same amount of traffic enters and leaves the roundabout.

4.3 Capacity

Roundabout approach capacity is dependent on the conflicting circulating flow and the roundabout's geometric elements.

The maximum flow rate that can be accommodated at a roundabout entry depends on two factors: the circulating flow on the roundabout that conflicts with the entry flow, and the geometric elements of the roundabout.

When the circulating flow is low, drivers at the entry are able to enter the roundabout without significant delay. The larger gaps in the circulating flow are more useful to the entering drivers and more than one vehicle may enter each gap. As the circulating flow increases, the size of the gaps in the circulating flow decrease, and the rate at which vehicles can enter also decreases. Note that when computing the capacity of a particular leg, the actual circulating flow to use may be less than demand flows, if the entry capacity of one leg contributing to the circulating flow is less than demand on that leg.

Roundabouts should be designed to operate at no more than 85 percent of their estimated capacity. Beyond this threshold, delays and queues vary significantly from their mean values.

The geometric elements of the roundabout also affect the rate of entry flow. The most important geometric element is the width of the entry and circulatory roadways, or the number of lanes at the entry and on the roundabout. Two entry lanes permit nearly twice the rate of entry flow as does one lane. Wider circulatory roadways allow vehicles to travel alongside, or follow, each other in tighter bunches and so provide longer gaps between bunches of vehicles. The flare length also affects the capacity. The inscribed circle diameter and the entry angle have minor effects on capacity.

As at other forms of unsignalized intersection, when traffic flows on an approach exceed approximately 85 percent of capacity, delays and queue lengths vary significantly about their mean values (with standard deviations of similar magnitude as the means). For this reason, the analysis procedures in some countries (Australia, Germany, and the United Kingdom), and this guide, recommend that roundabouts be designed to operate at no more than 85 percent of their estimated capacity.

As performance data become available for roundabouts designed according to the procedures in this guide in the United States, they will provide a basis for development of operational performance procedures specifically calibrated for U.S. conditions. Therefore, analysts should consult future editions of the *Highway Capacity Manual.*

4.3.1 Single-lane roundabout capacity

Exhibit 4-3 shows the expected capacity for a single-lane roundabout for both the urban compact and urban/rural single-lane designs. The exhibit shows the variation of maximum entry flow as a function of the circulating flow on the roundabout. The calculation of the circulating flow was described previously. The capacity forecast shown in the chart is valid for single-lane roundabouts with inscribed circle diameters of 25 m to 55 m (80 ft to 180 ft). The capacity forecast is based on simplified British regression relationships in Appendix A, which may also be derived with a gap-acceptance model by incorporating limited priority behavior.

Note that in any case, the flow rate downstream of the merge point (between the entry and the next exit) should not be allowed to exceed 1,800 veh/h. Exceeding this threshold may indicate the need for a double-lane entry.

Circulating flow should not exceed 1,800 veh/h at any point in a single-lane roundabout. Exit flows exceeding 1,200 veh/h may indicate the need for a double-lane exit.

The urban compact design is expected to have a reduced capacity, but has significant benefits of reduced vehicle speeds through the roundabout (per the German equations in Appendix A). This increases safety for pedestrians and bicyclists compared with the larger single lane roundabouts. Mini-roundabout capacities may be approximated using the daily maximum service volumes provided for them in Chapter 3, but in any case should not exceed the capacity of the urban compact design.

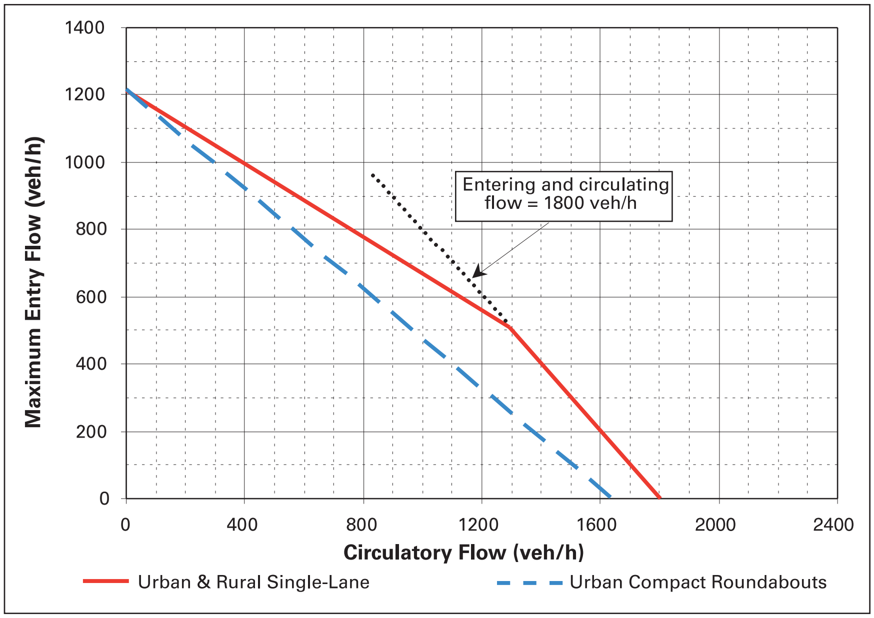


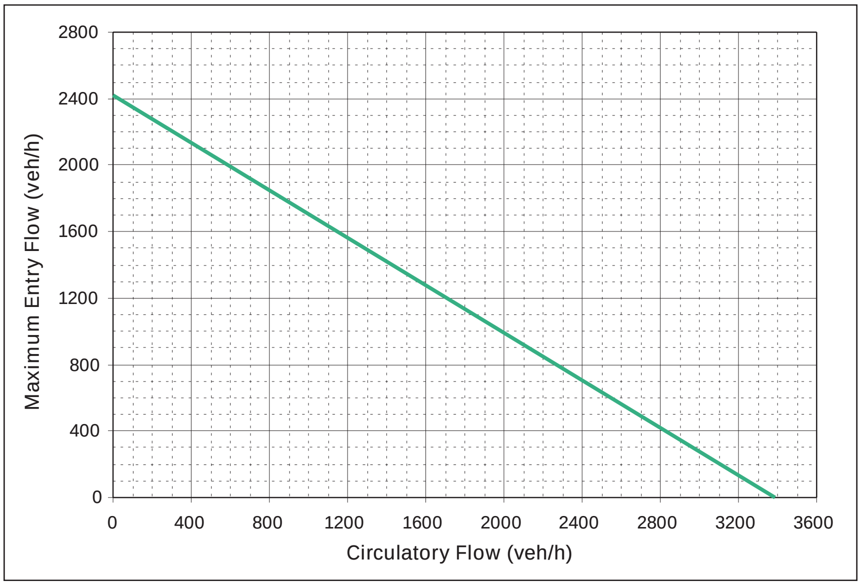
Exhibit 4-3. Approach capacity of a single-lane roundabout.

The slope of the upper line changes because circulating flow downstream from a roundabout entry should not exceed 1,800 veh/h.

4.3.2 Double-lane roundabout capacity

Exhibit 4-4 shows the expected capacity of a double-lane roundabout that is based on the design templates for the urban/rural double-lane roundabouts. The capacity forecast shown in the chart is valid for double-lane roundabouts with inscribed circle diameters of 40 m to 60 m (130 ft to 200 ft). The capacity forecast is based on simplified British regression relationships in Appendix A, which may also be derived with a gap-acceptance model by incorporating limited priority behavior. Larger inscribed diameter roundabouts are expected to have slightly higher capacities at moderate to high circulating flows.

Exhibit 4-4. Approach capacity of a double-lane roundabout.



When flared approaches are used, the circulatory road width must be widened.

See Appendix A for further information on the effects of short lanes at flared entries.

4.3.3 Capacity effect of short lanes at flared entries

By flaring an approach, short lanes may be added at the entry to improve the performance. If an additional short lane is used, it is assumed that the circulatory road width is also increased accordingly. The capacity of the entry is based on the assumption that all entry lanes will be effectively used. The capacity is given by the product of the appropriate factor in Exhibit 4-5 and the capacity of a two-lane roundabout in Exhibit 4-4. Refer to Appendix A for a derivation of these factors (9).

Number of vehicle spaces in the short lane, n_f	Factor (applied to double-lane approach capacity)
0 *	0.500
1	0.707
2	0.794
4	0.871
6	0.906
8	0.926
10	0.939

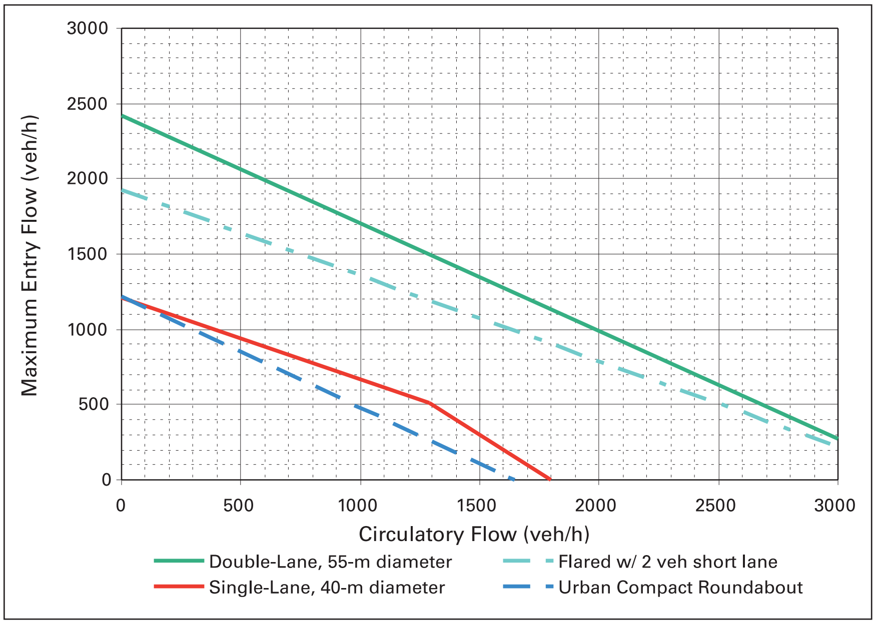
*Used for the case of a single lane entry to a double-lane roundabout.

Exhibit 4-5. Capacity reduction factors for short lanes.

The use of short lanes can nearly double approach capacity, without requiring a two-lane roadway prior to the roundabout.

4.3.4 Comparison of single-lane and double-lane roundabouts

Exhibit 4-6 shows a comparison of the expected capacity for both the single-lane and double-lane roundabouts. Again, it is evident that the number of lanes, or the size of the entry and circulating roadways, has a significant effect on the entry capacity.



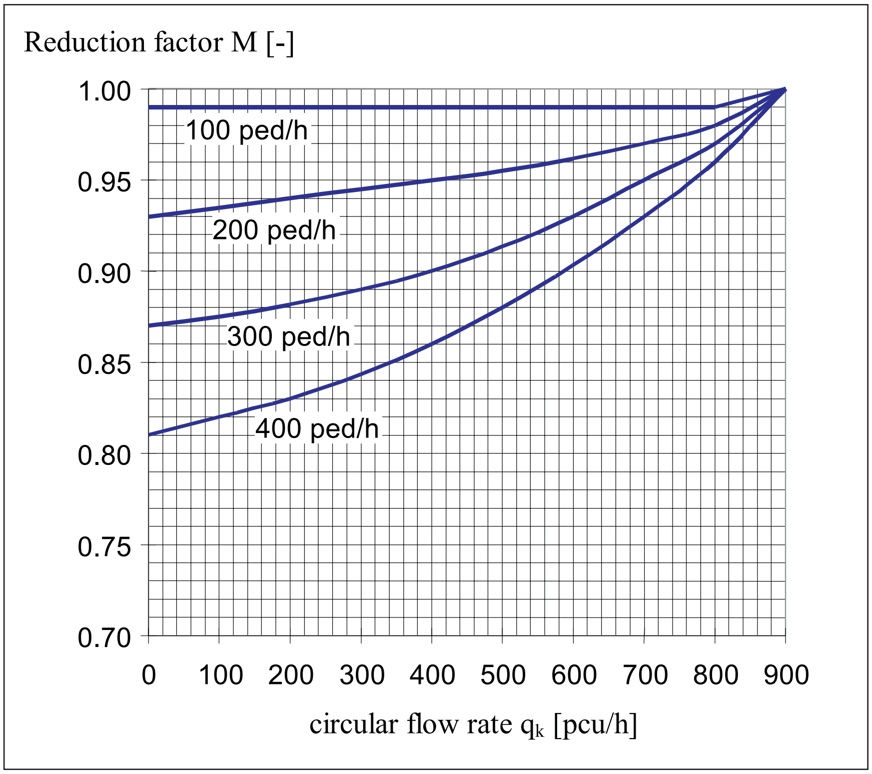
Source (10)

Exhibit 4-6. Capacity comparison of single-lane and double-lane roundabouts.

4.3.5 Pedestrian effects on entry capacity

Pedestrians crossing at a marked crosswalk that gives them priority over entering motor vehicles can have a significant effect on the entry capacity. In such cases, if the pedestrian crossing volume and circulating volume are known, the vehicular capacity should be factored (multiply by *M*) according to the relationship shown in Exhibit 4-7 or Exhibit 4-8 for single-lane and double-lane roundabouts, respectively. Note that the pedestrian impedance decreases as the conflicting vehicle flow increases. The *Highway Capacity Manual* (1) provides additional guidance on the capacity of pedestrian crossings and should be consulted if the capacity of the crosswalk itself is an issue.

Exhibit 4-7. Capacity reduction factor *M* for a single-lane roundabout assuming pedestrian priority.



Source: (10)

The effects of conflicting pedestrians on approach capacity decrease as conflicting vehicular volumes increase, as entering vehicles become more likely to have to stop regardless of whether pedestrians are present.

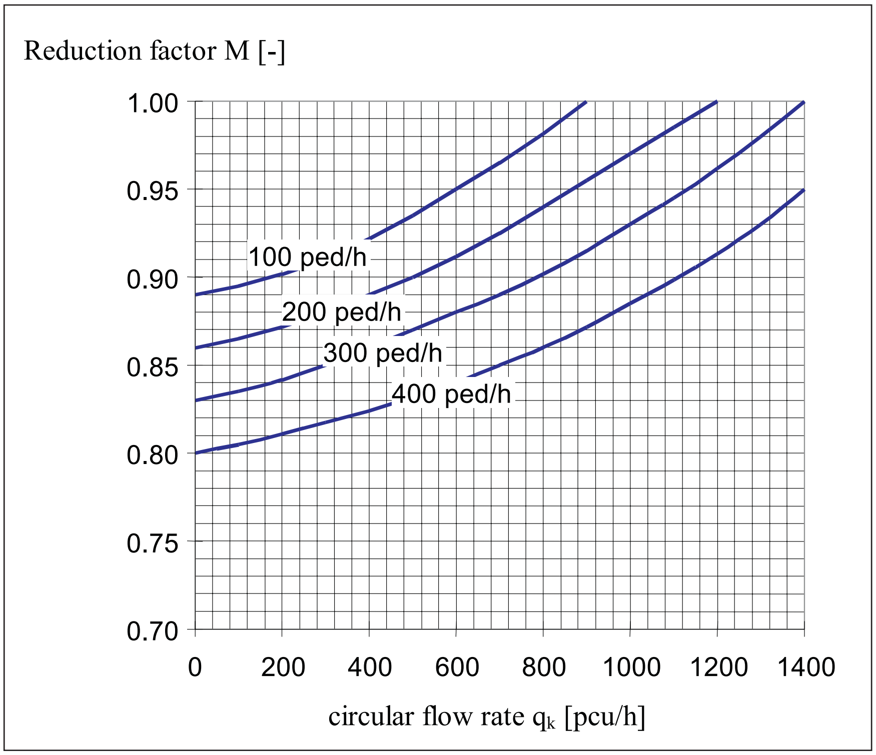


Exhibit 4-8. Capacity reduction factor *M* for a double-lane roundabout assuming pedestrian priority.

Source: (10)

4.3.6 Exit capacity

An exit flow on a single lane of more than 1,400 veh/h, even under good operating conditions for vehicles (i.e., tangential alignment, and no pedestrians and bicyclists) is difficult to achieve. Under normal urban conditions, the exit lane capacity is in the range of 1,200 to 1,300 veh/h. Therefore, exit flows exceeding 1,200 veh/h may indicate the need for a double-lane exit (11).

4.4 Performance Analysis

Three performance measures are typically used to estimate the performance of a given roundabout design: degree of saturation, delay, and queue length. Each measure provides a unique perspective on the quality of service at which a roundabout will perform under a given set of traffic and geometric conditions. Whenever possible, the analyst should estimate as many of these parameters as possible to obtain the broadest possible evaluation of the performance of a given roundabout design. In all cases, a capacity estimate must be obtained for an entry to the roundabout before a specific performance measure can be computed.

Key performance measures for roundabouts:

- **Degree of saturation**
- **Delay**
- **Queue length**

4.4.1 Degree of saturation

Degree of saturation is the ratio of the demand at the roundabout entry to the capacity of the entry. It provides a direct assessment of the sufficiency of a given design. While there are no absolute standards for degree of saturation, the Australian design procedure suggests that the degree of saturation for an entry lane should be less than 0.85 for satisfactory operation. When the degree of saturation exceeds this range, the operation of the roundabout will likely deteriorate rapidly, particularly over short periods of time. Queues may form and delay begins to increase exponentially.

4.4.2 Delay

Delay is a standard parameter used to measure the performance of an intersection. The *Highway Capacity Manual* (1) identifies delay as the primary measure of effectiveness for both signalized and unsignalized intersections, with level of service determined from the delay estimate. Currently, however, the *Highway Capacity Manual* only includes control delay, the delay attributable to the control device. Control delay is the time that a driver spends queuing and then waiting for an acceptable gap in the circulating flow while at the front of the queue. The formula for computing this delay is given in Equation 4-7 (12, based on 13; see also 14). Exhibit 4-9 shows how control delay at an entry varies with entry capacity and circulating flow. Each curve for control delay ends at a volume-to-capacity ratio of 1.0, with the curve projected beyond that point as a dashed line.

$$d = \frac{3600}{c_{m,x}} + 900T \times \left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{v_x}{c_{m,x}} - 1\right)^2 + \frac{\left(\frac{3600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{450T}} \right] \tag{4-7}$$

where: d = average control delay, sec/veh;
v_x = flow rate for movement x, veh/h;
c_{mx} = capacity of movement x, veh/h; and
T = analysis time period, h (T = 0.25 for a 15-minute period).

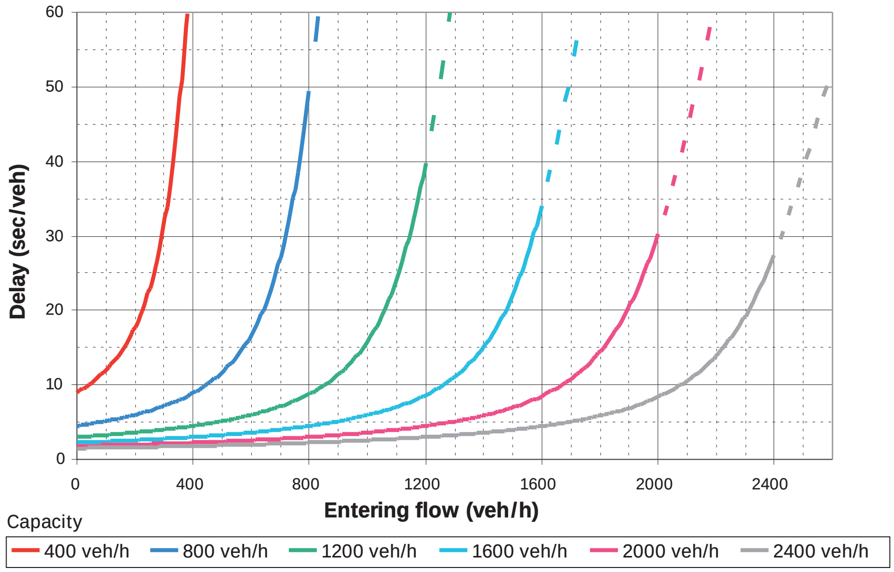


Exhibit 4-9. Control delay as a function of capacity and entering flow.

Note that as volumes approach capacity, control delay increases exponentially, with small changes in volume having large effects on delay. An accurate analysis of delay under conditions near or over saturation requires consideration of the following factors:

- *The effect of residual queues.* Roundabout entries operating near or over capacity can generate significant residual queues that must be accounted for between consecutive time periods. The method presented above does not account for these residual queues. These factors are accounted for in the delay formulae developed by Kimber and Hollis (15); however, these formulae are difficult to use manually.

- *The metering effect of upstream oversaturated entries*. When an upstream entry is operating over capacity, the circulating volume in front of a downstream entry is less than the true demand. As a result, the capacity of the downstream entry is higher than what would be predicted from analyzing actual demand.

For most design applications where target degrees of saturation are no more than 0.85, the procedures presented in this section are sufficient. In cases where it is desired to more accurately estimate performance in conditions near or over capacity, the use of software that accounts for the above factors is recommended.

Geometric delay is the additional time that a single vehicle with no conflicting flows spends slowing down to the negotiation speed, proceeding through the intersection, and accelerating back to normal operating speed. Geometric delay may be an important consideration in network planning (possibly affecting route travel times and choices) or when comparing operations of alternative intersection types. While geometric delay is often negligible for through movements at a signalized or stop-controlled intersection, it can be more significant for turning movements such as those through a roundabout. Calculation of geometric delay requires an estimate of the proportion of vehicles that must stop at the yield line, as well as knowledge of the roundabout geometry as it affects vehicle speeds during entry, negotiation, and exit. Procedures for calculating the number of stops and geometric delay are given in the Australian design guide (16).

4.4.3 Queue length

Queue length is important when assessing the adequacy of the geometric design of the roundabout approaches.

The average queue length (L vehicles) can be calculated by Little's rule, as shown in Equation 4-8 (17):

$$L = v \cdot d / 3600 \qquad (4\text{-}8)$$

where: v = entry flow, veh/h
d = average delay, seconds/veh

Average queue length is equivalent to the vehicle-hours of delay per hour on an approach. It is useful for comparing roundabout performance with other intersection forms, and other planning procedures that use intersection delay as an input.

For design purposes, Exhibit 4-10 shows how the 95th-percentile queue length varies with the degree of saturation of an approach (18, 19). The x-axis of the graph is the degree of saturation, or the ratio of the entry flow to the entry capacity. Individual lines are shown for the product of T and entry capacity. To determine the 95th-percentile queue length during time T, enter the graph at the computed degree of saturation. Move vertically until the computed curve line is reached. Then move horizontally to the left to determine the 95th-percentile queue length. Alternatively, Equation 4-8 can be used to approximate the 95th-percentile queue. Note that the graph and equation are only valid where the volume-to-capacity ratio immediately before and immediately after the study period is no greater than 0.85 (in other words, the residual queues are negligible).

$$Q_{95} \approx 900T\left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(1 - \frac{v_x}{c_{m,x}}\right)^2 + \frac{\left(\frac{3600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{150T}}\right]\left(\frac{c_{m,x}}{3600}\right) \tag{4-9}$$

where: Q_{95} = 95th percentile queue, veh,
v_x = flow rate for movement x, veh/h,
$c_{m,x}$ = capacity of movement x, veh/h, and
T = analysis time period, h (0.25 for 15-minute period).

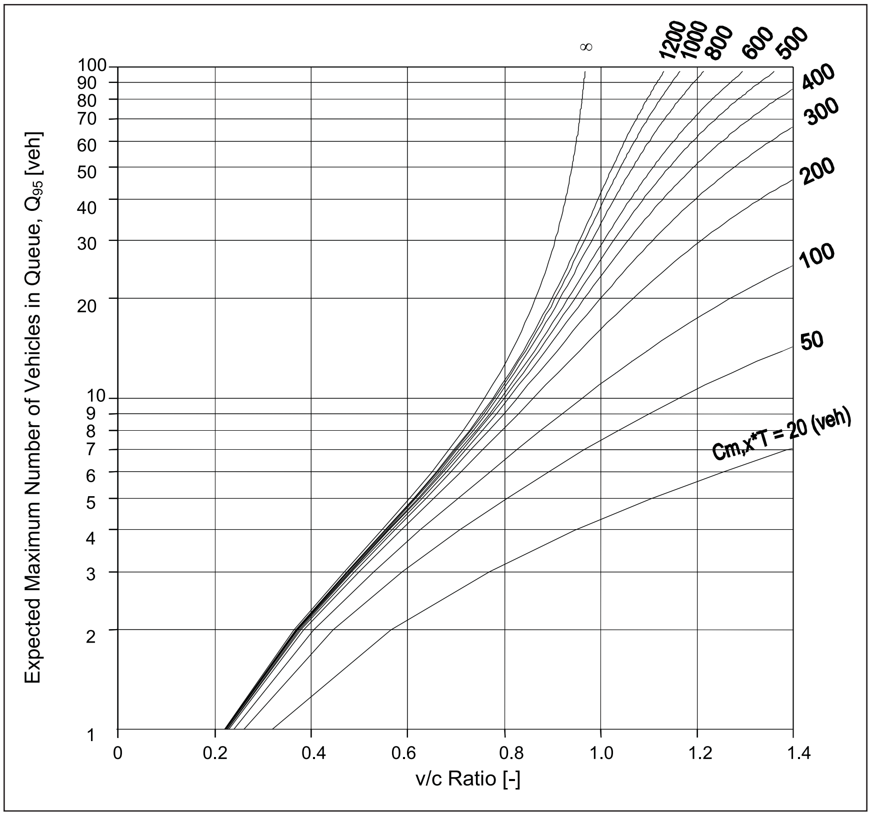


Exhibit 4-10. 95th-percentile queue length estimation.

Source: (19)

4.4.4 Field observations

The analyst may evaluate an existing roundabout to determine its performance and whether changes to its design are needed. Measurements of vehicle delay and queuing can be made using standard traffic engineering techniques. In addition, the analyst can perform a qualitative assessment of the roundabout performance. The following list indicates conditions for which corrective design measures should be taken (20). If the answers to these questions are negative, no corrective actions need be taken.

Points to consider for a qualitative assessment of roundabout performance.

- Do drivers stop unnecessarily at the yield point?
- Do drivers stop unnecessarily within the circulating roadway?
- Do any vehicles pass on the wrong side of the central island?
- Do queues from an external bottleneck back up into the roundabout from an exit road?
- Does the actual number of entry lanes differ from those intended by the design?
- Do smaller vehicles encroach on the truck apron?
- Is there evidence of damage to any of the signs in the roundabout?
- Is there any pedestrian activity on the central island?
- Do pedestrians and cyclists fail to use the roundabout as intended?
- Are there tire marks on any of the curb surfaces to indicate vehicle contact?
- Is there any evidence of minor accidents, such as broken glass, pieces of rim, etc., on the approaches or the circulating roadway?
- Is there any gravel or other debris collected in nontraveled areas that could be a hazard to bicycles or motorcyclists?
- Are the vehicle speeds appropriate?

4.5 Computer Software for Roundabouts

While the analytical procedures of different countries are not very complex, they are repetitive and time consuming, so most of these procedures have been implemented in software. A summary of current (as of 1999) software products and the analytical procedures that they implement is presented in Exhibit 4-11. The reader is also advised to consult the latest version of the U.S. *Highway Capacity Manual.* While the procedures provided in this chapter are recommended for most applications covered by this guide, models such as ARCADY, RODEL, SIDRA, KREISEL, or GIRABASE may be consulted to determine the effects of geometric parameters, particularly for multilane roundabouts outside the realm of this guide, or for fine-tuning designs to improve performance. Note that many of these models represent different underlying data or theories and will thus produce different results. Chapter 8 provides some information on microscopic simulation modeling which may be useful alternatives analysis in systems context.

Exhibit 4-11. Summary of roundabout software products for operational analysis.

Name	Scope	Application and Qualities (1999 versions)
ARCADY	All configurations	British method (50 percent confidence limits). Capacity, delay, and queuing. Includes projected number of crashes per year. Data were collected at extensive field studies and from experiments involving drivers at temporary roundabouts. Empirical relationships were developed from the data and incorporated into ARCADY. This model reflects British driving behavior and British roundabout designs. A prime attribute is that the capacities it predicts have been measured.
RODEL	All configurations including multiple roundabout interactions	British method (user-specified confidence limits). Capacity, delay, and queuing. Includes both an evaluation mode (geometric parameters specified) and a design mode (performance targets specified). Includes a crash prediction model. RODEL uses the British empirical equations. It also assists the user in developing an appropriate roundabout for the traffic conditions.
SIDRA	All configurations and other control types	Australian method, with analytical extensions. Capacity, delay, queue, fuel, and environmental measures. Also evaluates two-way stop-controlled, all-way stop controlled, and signalized intersections. It also gives roundabout capacities from U.S. HCM 1997 and German procedures. SIDRA is based on gap acceptance processes. It uses field data for the gap acceptance parameters to calibrate the model. There has been limited field evaluation of the results although experience has shown that the results fit Australian and U.S. single-lane (21) roundabout conditions satisfactorily. An important attribute is that the user can alter parameters to easily reflect local driving.
HCS-3	Single-lane roundabouts with a limited range of volumes	U.S. HCM 1997 method. Limited to capacity estimation based on entering and circulating volume. Optional gap acceptance parameter values provide both a liberal and conservative estimate of capacity. The data used to calibrate the models were recorded in the U.S. The two curves given reflect the uncertainty from the results. The upper-bound average capacities are anticipated at most roundabouts. The lower bound results reflect the operation that might be expected until roundabouts become more common.
KREISEL	All configurations	Developed in Germany. Offers many user-specified options to implement the full range of procedures found in the literature from U.S. (including this chapter), Europe, Britain, and Australia. KREISEL gives the average capacity from a number of different procedures. It provides a means to compare these procedures.
GIRABASE	All configurations	French method. Capacity, delay, and queuing projections based on regression. Sensitive to geometric parameters. Gives average values.

4.6 References

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