4.0 STRUCTURE

Table of Contents

4.0 Structure

4.a General Introduction

4.a.1 Introduction

A. The purpose of the Emaar Structural design guidelines is to provide consistent criteria for design issues that may arise across Emaar residential developments. The criteria have been developed with reference to local and international experience as well as international design standards. This guideline does not cover every design situation. Where specific design criteria are in question and are not included in this document, it is the responsibility of the project consultant to propose, discuss and agree on the respective issue with Emaar.

B. These guidelines do not relieve the project design consultant of responsibility for accurately determining capacities, loads, and sizes or complying with local authority requirements. They can be used for initial design guidance, act as good practice guidelines for the structural design may be interpreted as minimum design requirements and may act as a 'reminders' or 'cautionary watch it notes' for specific design issues.

4.a.2 Objectives

A. The main objectives of this document shall be as follows:

1. Confirm the design parameters to be used for the structural design of buildings in Dubai, UAE.
2. Provide an economic structural design by using:
    a. Reduced Superimposed dead loads
    b. Preference for wind tunnel studies where applicable
    c. Appropriate seismic analysis
    d. Others
3. Improve floor plate efficiency and overall constructability
4. Outlines with design guidelines/codes to be adhered to during the design process.
5. Discussion on uniform and consistent column grids, use of transfer elements etc.

B. It is expected that this document will form the basis for selecting a structural layout and design values for the basis of design criteria report developed for each project by the design consultants.

4.a.3 General

A. This guideline shall be used as a generic document for both uses by Consultants towards their design process for any Residential Building project under EMAAR. This guideline has to be read in conjunction with all related International standards, codes for different disciplines along with the respective local authority regulations.

B. State that all materials, equipment, and workmanship must conform to all pertinent codes, laws, ordinances, and regulations of all bodies having jurisdiction.

C. The consultants shall state which codes and standards apply: ACI, UBC, BS, ASCE, Etc. by title and document number.

4.a.4 Definition of Terms

A. The below provides a description of the most used words in the structural design guidelines document.

Guidelines – refers to this document "EMAAR Structural Design Guidelines"

Consultant – refers to the main structural engineering design consultant appointed by EMAAR at any given phase of the project

Shear wall – a wall designed to resist lateral forces parallel to the plane of a wall

Bearing Wall- a shear wall designed to resist as main LFRS (Lateral Force Resistance System) against of in-plane and out-plane forces

Backstay Effect – The set of lateral forces developing within a podium structure to equilibrate the lateral forces and moment of a tower extending above the podium structure. This condition is common to tall core wall buildings in which the core extends into a stiff basement structure braced by stiff basement walls around the perimeter.

Core – a collection of walls, typically located at the middle of the floor plate of a multi-story building which houses lift shafts, stairs, and building services and riser shafts

Core wall – a wall element that is part of a core as defined above

Coupled Wall – A series of wall segments coupled and connected with coupling beam (link beam)

Coupling beam – A beam/spandrel component connect to piers in order coupling two walls for more efficiency against of lateral force

Transfer structure (or element) – any beam, slab or truss structure used to redirect the vertical gravity or lateral load path of upper stories to the vertical structure of the lower stories

Shamal winds – hot and dry, dusty wind from the north or northwest in Iraq, Iran and the Arabian Peninsula that causes great dust storms

Wall – defined by IBC 2012 as a vertical structural element having a length greater than 3 times its thickness

Monotonic loading – Loading of a structural component in which the displacement increases monotonically without unloading or reloading.

Nominal Strength –The strength of an element, calculated using specified material properties and the strength formulation specified by the standard of the applicable material, before application of a resistance (strength reduction) factor.

Residual Story Drift Ratio – The value of story drift ratio at a location in a structure at rest, following response to earthquake motion.

Return Period – The average time span between shaking intensity that is equal to or greater than a specified value, also known as the recurrence interval; the annual frequency of exceeding a given intensity is equal to the reciprocal of the return period for that intensity.

Service-Level Earthquake Shaking – Ground shaking represented by an elastic, damped, acceleration response spectrum that has a return period of 43 years, approximately equivalent to a 50% exceedance probability in 30 years.

Site Response Analysis – Analysis of wave propagation through a soil medium used to assess the effect of local geology on the ground motion.

Story Drift Ratio – The difference, at a specific instance of time, in lateral deflections at two adjacent horizontal levels divided by the vertical distance between the levels, commonly taken along principal axes of the building.

Transient Story Drift Ratio – The maximum absolute value of story drift ratio that occurs during a single response history analysis.

Uniform Hazard Spectrum – A site-specific, acceleration response spectrum constructed such that the ordinate at each natural period has the same exceedance probability or average return period.

Orthogonal (bi-directional) Effects - The earthquake load effects on structural elements common to the lateral-force-resisting systems along two orthogonal axes.

4.a.5 Abbreviations & Definitions

Table 1: Abbreviations & Definitions Table

Abbreviations	Definitions
ACI	American Concrete Institute
AAC	Autoclaved Aerated Concrete
ASCE	American Society of Civil Engineers
AISI	American Iron and Steel Institute
AISC	American Institute of Steel Construction
ASTM	American Society for Testing and Materials
AWS	American Welding Society
ATC	Applied Technology Council
CEB	Comité européen du béton (European Committee for Concrete)
CTBUH	Council on Tall Buildings and Urban Habitat
DBE	Design Base Earthquake, defined by ASCE 7-05/UBC97 as the earthquake effects that are two-thirds of the corresponding Maximum Considered Earthquake (MCE_R) effects
DDA	Dubai Development Authority
DM	Dubai Municipality
DMD	Dubai Municipality Datum
FEMA	Federal Emergency Management Agency
FIP	Fédération Internationale de la Précontrainte (International Federation for Prestressing)
GRP	Glass Reinforced Plastic
HSS	Hollow Structural Section
IBC	International Building Code
ISO	International Organization for Standardization
LLRSS	Lateral Load-Resisting Structural System
LATBSDC	Los Angeles Tall Building Structural Design Council
MCE_R	Risk-targeted Maximum Considered Earthquake
MEP	Mechanical, Electrica, and Plumbing
MWFRS	Main Wind-Force Resisting System
NEHRP	National Earthquake Hazard Reduction Program
PAS	Publicly Available Specifications
PBD	Performance-Based Design
PGA	Peak Ground Acceleration
PSHA	Probabilistic Seismic Hazard Analysis
PS	Punching Shear
RHS	Rectangular Hollow Section
$RotD_{D50}$	Ground motions oriented so as to produce a geometric mean response

Abbreviations	Definitions
$RotD_{D100}$	Ground motions oriented so as to produce maximum response
SCI	Steel Construction Institute
SHS	Square Hollow Section
SJI	Steel Joist Institute
SLE	Service-Level Earthquake
SLS	Serviceability Limit State
SSI	Soil-Structure Interaction
TBI	Tall Building Initiative – PEER Berkeley
Trakhees	Department of Planning & Development Ports, Customs & Freezone Corporation
TR	Technical Report
UBC	Uniform Building Code
ULS	Ultimate Limit State
VLRSS	Vertical Load-Resisting Structural System

4.a.6 Reference Documents

The following documents shall be used in addition to these Guidelines:

- Geotechnical Reports (refer to Chapter 7)
- Wind Tunnel Report (if commissioned by EMAAR, refer to section 4.2.2)
- EMAAR Standards Structural Modelling Guidelines

Note: Engagement with a wind tunnel consultant for structural wind tunnel testing is generally recommended.

Some projects may have specific design guidelines and as such should be also considered in addition to the above.

4.a.7 Design Standards

Every effort has been made to ensure that these guidelines cover all local authority requirements, principally those of DM & DDA, and that compatible international design codes of practice are referred to. However, it is the consultant's responsibility to ensure that all current local authority requirements are identified and considered in the design unless otherwise agreed with Emaar and the approving authority.

The 2009 edition of the International Building Code (IBC 2009) is specified herein as the overarching design code reference, and all other code editions specified are those that are compatible with this edition of IBC. The selection of IBC 2009 as the overarching design code has been made due to the requirements stated in DM Circular 37, which specifies the use of ASCE 7 for wind purposes, and for seismic loads. It is to be noted that ASCE 7-05 and 3-second gust speeds are not compatible with more recent versions of IBC than the 2009 edition, but the consultant may discuss and agree on the use of more recent versions of ASCE 7, and updated reference wind speeds, with DM if necessary. In such cases, it should be ensured that the relevant, compatible IBC version is then used as the overarching code, and all other design codes specified are compatible with this.

Further consideration needs to be given to the use of ASCE 7 for the derivation of seismic forces as specified by DM Circular 37. Use of codes other than ASCE 7, together with suitable adopted parameters, needs to be agreed with DM in advance.

In general terms, the structure shall be designed to meet or exceed the minimum requirements of the following codes:

Building Codes	• ASCE 7 • 2009 International Building Code (IBC 2009)

The structure shall be designed to meet or exceed the minimum requirements of the following reference standards, as modified by the Building Code:

"Note: Using the latest edition of below codes or standard has no objection if load effect due to load combination (in the lateral wind / seismic / Tsunami) based on IBC2009/UBC97 / ASCE7-05 unless the new criteria of loading will be announced by Authority"

Steel	AISC 360-05/10/16): "Specification for Structural Steel Buildings," by American Institute of Steel Construction AISC 341-05/10/16: "Seismic Provisions for Structural Steel Buildings", by American Institute of Steel Construction AISC 358-05/10/16: "Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications ", by American Institute of Steel Construction AISC 303-05/10/16: "Code of Standard Practice for Steel Buildings and Bridges", by American Institute of Steel Construction
Welding	AWS D1.1M-2015: "Structural Welding Code - Steel," by American Welding Society AWS D1.4M-2018: "Structural Welding Code – Reinforcing Steel," by American Welding Society AWS D1.2M-2014: "Structural Welding Code – Aluminum," by American Welding Society

Concrete	ACI 318M-08: "Building Code Requirements for Reinforced Concrete," 2011, by the American Concrete Institute, ACI318M-11/14 can be addressed if the note is above considered. ACI318-19 to be used in conjunction with ASCE7-16 Load combination if the Authority accepts the terms of lateral loading of it. (Reduction in shear capacity is considered in case loading criteria and combination are based on ASCE7-16 and LLRSS is not designed based on ASCE7-16) ACI 435 – Control of Deflection in Concrete Structures ACI 224.2R-92 Cracking of Concrete Members in Direct Tension ACI 209.2R-08 Guide for Modelling and Calculating Shrinkage and Creep in Hardened Concrete ACI 216.1M-07/TMS-216-07 Code Requirements for Determining Fire Resistance of Concrete and Masonry Construction Assemblies ACI 224-01 Control of Cracking in Concrete Structures ACI 315-99 Details and Detailing of Concrete Reinforcement ACI 224.3R-95 Joints in Concrete Construction ACI 207.2R Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete. ACI SP240- Performance-Based Design of RC Buildings for Wind Loads – Overview and Issues
Precast Concrete and Pre-stressed concrete	ACI 318M-08: "Building Code Requirements for Reinforced Concrete," 20081, by the American Concrete Institute ACI318M-11/14 can be addressed if the note is above considered. MNL-120-04 "PCI Design Handbook," Sixth Edition, by Precast/Prestressed Concrete Institute
Masonry	ACI 530-08: "Building Code Requirements for Masonry Structures," 2008, by the American Concrete Institute ACI 530.1-08: "Specification for Masonry Structures," 2008, by the American Concrete Institute
Cold-formed steel	AISI S100-07/SI-10: "North American Specification for the Design of Cold-Formed Steel Structural Members, with Supplement 1", 2007 by American Iron and Steel Institute
Aluminum	AA ADM 1-2010: "Specifications for Aluminium Structures," by the Aluminium Association
Open Web Steel Joists	SJI -1994: "Standard Specification, Load Tables and Weight Tables for Steel Joists and Joist Girders," 1994, by the Steel Joist Institute
Wind Loads	ASCE 7: "Minimum Design Loads for Buildings and Other Structures", by American Society of Civil Engineers
Seismic Loads	ASCE 7 under consultation and agreement with Dubai Municipality

The following references may be used as supplementary guidelines for structural design:

1. Sulfate and acid resistance of concrete in the ground. BRE Special Digest 1 (2005)
2. Reinforced Concrete Designers Handbook, Tenth Edition, Charles E Reynolds and James C Steedman, E + FN Spon, 1996.
3. Seismic Design of Reinforced Concrete Buildings by Prof Jack Moehle
4. ASCE41-17 – Seismic Evaluation and Retrofit of Existing Building
5. fib Model Code for Concrete Structures 2010
6. NEHRP Seismic Design Technical Brief No. 3 (Seismic Design of Cast-in-Place Concrete Diaphragms, Chords, and Collectors – A Guide for Practicing Engineers)
7. NEHRP Seismic Design Technical Brief No. 7 (Seismic Design of Reinforced Concrete Mat Foundations A Guide for Practicing Engineers)
8. 2015 NEHRP Recommended Seismic Provisions: Design Examples
9. PTI sixth Edition Handbook
10. Steel, Concrete and Composite Design of Tall Buildings, Bungale S Taranath, Second Edition, 1997.
11. TR43 Post-tensioned concrete floors. Design handbook (Concrete Society, First Edition) Note – second edition (2005) is for Eurocodes.
12. TR49 Design Guidance for High Strength Concrete (Concrete Society, 1998)
13. TR51 Guidance on the use of stainless steel reinforcement (Concrete Society, 1998)
14. SCI P354 Design of Floors for Vibration: A New Approach
15. Column Shortening in Tall Buildings. Prediction and Compensation. M Fintel, S.K. Gosh, and H Iyengar
16. UBC 1997 Uniform Building Code. (seismic design)
17. CEB-FIP model code 1990: design code
18. Reinforced Concrete Structures, R.Park and T.Pauley, Wiley, 1975
19. Seismic Design of Reinforced Concrete and Masonry Buildings, T.Paulay, M.J.N.Priestley. Wiley,1992
20. Design of Pre-stressed Concrete Structures-Second Edition, T.Y.Lin, Wiley, 1966
21. CIRIA C577 – Guide to the Construction of Reinforced Concrete in the Arabian Peninsula, 2002
22. AISC Design Guideline Series 1~33 For steel structure
    a. Design Guide 1: Base Plate and Anchor Rod Design (Second Edition)
    b. Design Guide 2: Design of Steel and Composite Beams with Web Openings
    c. Design Guide 3: Serviceability Design Considerations for Steel Buildings (Second Edition)
    d. Design Guide 4: Extended End-Plate Moment Connections Seismic and Wind Applications (Second Edition)
    e. Design Guide 5: Design of Low- and Medium-Rise Steel Buildings
    f. Design Guide 6: Load and Resistance Factor Design of W-Shapes Encased in Concrete
    g. Design Guide 8: Partially Restrained Composite Connections
    h. Design Guide 9: Torsional Analysis of Structural Steel Members

i. Design Guide 10: Erection Bracing of Low-Rise Structural Steel Frames
j. Design Guide 11: Vibrations of Steel-Framed Structural Systems Due to Human Activity (Second Edition)
k. Design Guide 12: Modification of Existing Steel Welded Moment Frame Connections for Seismic
l. Design Guide 13: Wide-Flange Column Stiffening at Moment Connections
m. Design Guide 14: Staggered Truss Framing Systems
n. Design Guide 16: Flush and Extended Multiple-Row Moment End-Plate Connections
o. Design Guide 19: Fire Resistance of Structural Steel Framing
p. Design Guide 20: Steel Plate Shear Walls
q. Design Guide 21: Welded Connections--A Primer for Engineers, Second Edition
r. Design Guide 22: Facade Attachments to Steel-Framed Buildings
s. Design Guide 23: Constructability of Structural Steel Buildings
t. Design Guide 24: Hollow Structural Section Connections
u. Design Guide 25: Frame Design Using Web-Tapered Members
v. Design Guide 28: Stability Design of Steel Buildings
w. Design Guide 29: Vertical Bracing Connections--Analysis and Design
x. Design Guide 31: Castellated and Cellular Beam Design
y. Design Guide 32: Design of Modular Steel-Plate Composite Walls for Safety-Related Nuclear Facilities
z. Design Guide 33: Curved Member Design
aa. Design Guide 34: Steel-Framed Stairway Design

23. Time-Dependent Behaviour of Concrete Structures, By Raymond Ian Gilbert
24. Design of Low-Rise Reinforced Concrete Building, Based on the 2009 IBC by David A. Fanella

4.b BIM Requirements

Emaar fully supports the Dubai Municipality BIM mandates contained in circular 196 (2013) and circular 207 (2015).

To efficiently deliver against those requirements the Client supports the strategy as defined in PAS 1192-2 released in 2013 and other related documentation. This is considered as a global best practice available within the industry and is equivalent to level 2 BIM adoption in the UK.

4.b.1 PAS 1192-2

The key eight facets of this standard are recognized as:

1. A well-defined information management process (PAS 1192-2)
2. A holistic lifecycle approach, considering each stage of the project from inception to use (PAS 1192-3)
3. Standard formats for information exchange (including, but not limited to COBie)

4. Management of information security risks for sensitive assets (PAS 1192-5)
5. Object-based modeling with clearly defined Levels of Detail and Information for each modeled element, at each design stage (utilizing both clients defined and in-house Model Production & Delivery Tables (MPDT))
6. Consistent methods for naming and classification (including, but not limited to UniClass, UniFormat & the BS 8541 series)
7. Early engagement of clients, operators, and occupants (BS 8536-1, Government Soft Landings, and Post Occupancy Evaluation)
8. A contractual basis for BIM (including, but not limited to the CIC BIM Protocol)

Successful BIM projects can only truly be delivered when the Client and cohesive design team work harmoniously, fully utilizing the roles, processes, and documentation such as Employers Information Requirements (EIR) and BIM Execution Plans (BEP) prescribed by PAS 1192 and associated standards.

The integrity of any digital project is only as strong as its weakest link, without clear overall holistic direction specified within the EIR and BEP and associated standards, it is difficult to steer a project to an optimum outcome from within a single discipline.

Issues with technology, although often easily resolved, still need to be considered at the earliest stage and mitigated with plans that important work in line with the capabilities of the entire project team.

Key advantages of adopting a PAS 1192-2 Implementation are:

- The production of Correct, Complete, Coordinated and Consistent project information through the provision of project roles with appropriate levels of authority and the ability to enforce the digital requirements of the Client
- The ability to avoid scope gaps by openly engaging with all project stakeholders using consistent templates, standards and classification systems
- An enhanced design due to an improved ability to rapidly iterate design options using computational engineering techniques and early engagement with owners, operators, and occupants
- The expedited creation and ease of utilization within core modeling applications of accurate and detailed records of the existing condition of development, by using enhanced scanning techniques (point clouds)
- An improved ability to communicate with peers and clients in a formal manner through the use of issue tracking tools and databases
- An improved ability to communicate with both technical and non-technical stakeholders by using advanced visualization techniques including, CGI, real-time animations and virtual reality
- The provision of a robustly and demonstrably coordinated design, allowing the contractor/manufacturer/fabricator to price and undertake their works as efficiently as possible
- An ability to trend project costs at close intervals

- Improved logistics management through the inclusion of construction phasing parameters
- Inclusion of additional opportunities for project review at early stages providing the ability to make changes at a point in the project's lifecycle where that change is both possible and cost-effective

4.b.2 BIM Kick-off meeting

At the commencement of any individual project a multi-disciplinary BIM specific Kick-Off Meeting must be held, where all the relevant documentation to be implemented shall be reviewed and agreed.

4.b.3 BIM workshops and reviews

An inclusive approach to client and project team interaction throughout the development of the Building Information Modelling process is critical, including inviting all parties to attend regular BIM Strategy Workshops and Reviews. This enables informed decisions to be made at key points in the earlier stages of the building design process, providing cost-effective, efficient mitigation of the impact of potential design issues and/or change. This allows the Client to fully achieve the strategic objectives for both product delivery and operational management.

4.c Design Deliverables

The list below provides a generic list of structural design deliverables to be produced by the design consultant; Emaar may further refine the requirements based upon specific project needs. The design consultant shall confirm these requirements with Emaar at the beginning of the design. Any content deemed not applicable to the specific project may be omitted from the list below subject to a mutual agreement with Emaar.

4.c.1 Concept Design

- Concept Design Report1
- Steps for the basis of work like Soil Test, Wind Engineering (if required), Special Sub-Consultancy (ie. Geotechnical Engineering), Third Party and etc.
- Design Criteria / Basis of Design6
- Sketches of typical plans/column grids/sections
- Steps needed to achieve desired green building ratings as per the requirements of AL SAFAT-Dubai Green Building Requirements

4.c.2 Schematic Design

- Schematic Design Report2
- Schematic Analysis Model
- Reissue Design Criteria report with updates (if any)
- Schematic Scale Drawings Including
- Excavation
- Foundation

- Floor general arrangements plans
- Typical details
- Wind tunnel tender specification
- Wind tunnel tender return report
- Ground Investigation Tender specification
- Draft structural specifications
- Pile specifications3
- PTP test specification3
- Pile load schedule3
- Revit model – co-ordination
- Enabling works authority submission (confirm with EMAAR)
- Preliminary calculations for key elements

4.c.3 Design Development

- Reissue Design Criteria report with updates (if any)
- Outline structural specification
- Movement and tolerances draft report4
- Design Development Technical Report5
- Scale Drawings including
    - Foundation plans
    - Piling drawings (if present)
    - Floor framing layout
    - Wall Elevations
    - Floor reinforcement intent layout
    - Columns schedule- intent
    - Beams schedule- intent
    - Link beams schedule- intent
    - Core plans
    - Typical structural details
    - Steelwork main elevations
    - Loading plans
    - General notes
- Calculation submission for authority approval
- Structural analytical models
- Revit model – co-ordination

4.c.4 Tender Documentation

- Relevant input into General and Particular Conditions of Contract,
- Drawings,
- Reports (design criteria, movement, and tolerance, etc.)

- Specifications.

4.c.5 Construction Documentation

- Final Structural specifications including, but not limited to, the following:
  - Excavation and filling
  - Earthworks
  - Waterproofing
  - Termite control
  - Concrete
  - Steelwork
  - Post-tensioning (if applicable)
  - Precast Design and Drawings
- Movement and tolerances final report
- Scale Drawings including, but not limited to:
  - Excavation drawings
  - Foundation plans
  - Piling drawing
  - Foundations' typical details
  - Foundations' reinforcement details
  - Walls sections
  - Walls reinforcement sections
  - Floors' GAs
  - Floors' reinforcement drawings
  - Reinforcement details (Specific RC details not covered in typical structural details)
  - Main core walls elevations
  - Columns schedule
  - Beams schedule
  - Link beams schedule
  - Core plans
  - Core reinforcement plans
  - Typical structural details
  - Stair drawings
  - Steelwork main elevations
  - Steelwork connections' intent and design forces
  - Loading plans
  - General notes
- Revit model – co-ordination

Notes:

1. The concept design report shall include the following as a minimum:
    a. a description of main structural systems proposed for foundations, VLRSS, LLRSS, etc. and options considered along with recommendations for the schematic design
    b. highlight unusual features or requirements for elements such as transfer and other irregular structural systems.
    c. Highlight any project risks.
    d. Highlight deviation from design approach recommended in this document and obtain approval on such deviation.
2. In subsequent design reports the contents of the concept design report shall be further developed and further detail added to reflect the progress in design.
3. As agreed with EMAAR or as per project requirements.
4. The movement and tolerance report is a document highlighting the details of building movements by the structural engineer which can be used by the designers of other building elements such as façade designer. If realistic estimates of building movement are not available at a specific design stage then provide limits of deflection and drift for which structure would be designed. The contents of the movement and tolerance report shall as a minimum contain the following:
    a. Lateral movement due to the wind and seismic actions
    b. Lateral gravity movement
    c. Vertical gravity movement of beams and slabs including short and long-term effects
    d. Movement at movement joints with recommended building separation (if applicable)
    e. Foundation settlements
    f. Short and long term elastic shortening, shrinkage, and creep movement of columns and walls
    g. Building tolerances to be considered in the design and detailing
5. This item is not typically required, but may be necessary where unusual structural features are present or otherwise required by EMAAR. The Consultant should discuss and with EMAAR to agree on the requirements for a Design Development Technical report.
6. The Design Criteria basis shall include the following as a minimum.
    a. Brief Description of the Project with a description of the main structural system.
    b. Applicable Standards and Codes
    c. Properties of Structural Material used
    d. Applicable loads. Gravity, Lateral, Temperature, Hydrostatic, Earth Pressure, etc.
    e. Foundation Design Criteria.
    f. Performance Criteria.
    g. Fire resistance.
    h. Construction methodology such as pour strips, expansion and construction joints, etc.

4.d Design Approach

The recommendations below are based upon a typical residential low-rise, medium-rise and typical tall building with approximately 40 - 70 stories of with a number of levels of parking basement and above-ground podium with a generally uniform grid of columns from the roof to the foundation level, with centrally located core walls around elevator bank and stairs servicing various floors.

These serve as general guidance and outline a preferred approach of Emaar to emphasize the most economy of the designed building structure. These recommendations have been put together with a view to maximizing floor plate efficiency, structural economy and optimize floor to floor height.
It is noted that not all buildings are the same, and suggested systems and criteria may not be appropriate in all cases. Emaar will consider deviation from these guidelines on a case by case basis with appropriate justification provided by the design consultant.

In all cases, it should be ensured that design criteria are being coordinated with the architecture and other disciplines, particularly in terms of floor finishes and specification of the partitions, etc., but exceptions to criteria contained herein must first be approved by Emaar.

This guideline document is NOT meant to qualify, replace or alter local regulations, design codes and sound engineering judgment.

4.d.1 Preferred Structural Systems

4.d.1.1 Column Grid and Floor Plate

The positions of columns within a floor plan are generally derived through architectural consideration, while the size and uniformity of the column grid had an appreciable impact on many aspects of the structure.

While acknowledging that there is a high degree of variability in choosing a column grid driven by plot size, floor plan, and building shape, a rationalisation of the column grid is desirable, and coordination between the Architect and the Structural Designer during the preliminary phase is crucial.

EMAAR's recommendation for the column grid dimension is the use of a regular grid of approximately 8.5m x 8.5m in dimension. This grid spacing can accommodate three car parking spaces between columns, and a regular drive-way. The 8.5m column grid will also allow reasonable column-free residential, office, and retail space.

Generally, a flat slab without drop beams is preferred for typical basement, podium and tower floor plates. In some instances, a thicker slab band may be needed around edges to carry heavier cladding load and to control edge deflection, particularly adjacent to balconies. Proprietary flat slab formwork systems typically accommodate approximately 150 mm height adjustment, so down-stands should be limited to this value where possible to ensure quick removal of formwork to enable quicker floor cycles. Further guidance in this regard may be obtained from formwork contractors or likely main contractors through coordination with Emaar.

It is generally preferable to design floor slabs with punching shear reinforcement, within practical limits, rather than using a thicker slab, column head or a drop panel. Nevertheless, the design consultant shall study cost, time and design implications of all relevant, practical options.

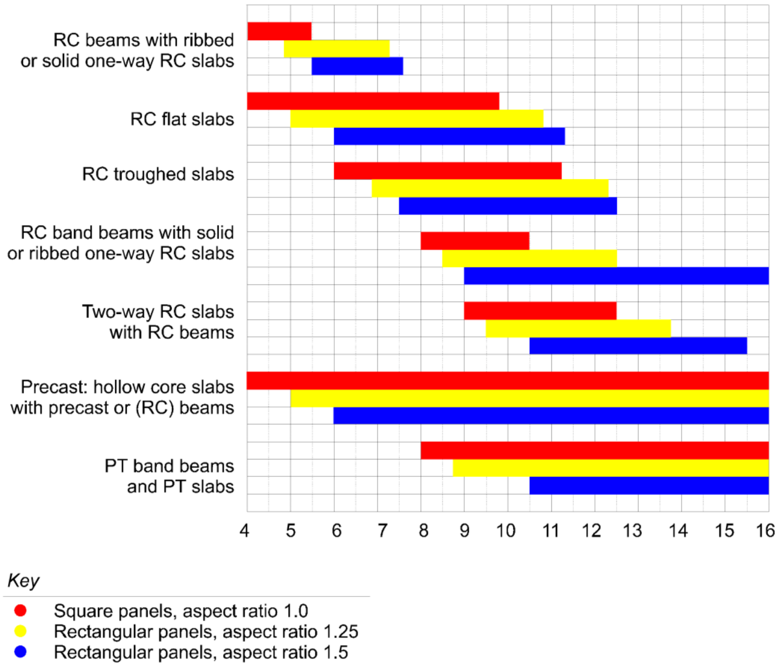
Due to the arrangement of apartments, it is generally expected that the main columns will be placed along partition walls between units. It is expected that the consultant shall study the impact of different column lengths to ensure the optimum balance of column size and slab thickness. The total volume of concrete and weight of reinforcement for columns and slabs and foundations shall be considered for options and the most economical overall solution adopted.

An 8.5m span generally affords a modest reinforced concrete flat slab thickness of between 225 to 275mm based upon normal light parking level loads and typical residential floor loads. Floors subject to heavier loading, such as MEP and podium floors, may have thicker slabs or drop beams as required, subject to comments made earlier in this section. Thinner floors are expected if a -post-tensioned slab system is employed. The designer shall perform an overall cost-benefit study by engaging with the project quantity surveyor.

Flat slab soffits increase the efficiency of formwork erection, laying of reinforcement, etc., resulting in an overall improvement in the economy of structure. The easier layout of MEP routing and ducts below the slab also improves efficiency and installation durations.

For the design of slabs (except the HCS type), the connection at core walls shall be assumed fixed for both strength design as for serviceability checks (i.e. deflection). Resultant pull-out bars at the core interface shall be designed to cover both strength and serviceability requirements of these two cases. Pull-out bars from the core should ideally be limited to T12 or smaller for ease of construction purposes, but larger bars can be used effectively with couplers and keyways.

Concrete Floor Slabs: Typical Econmic Span Ranger



Note
all subject to market conditions and project specific requirements
RC=Reinforced concrete PT=Post-tensioned concrete

For initial design in the schematic, above figure (ref. ARUP), can be addressed unless the justification provides by the consultant.

4.d.1.2 Vertical load resisting structural system

It is expected that the vertical load resisting system shall typically consist of reinforced concrete columns and a central concrete core.

The size and shape of the columns shall complement the architectural layout while ensuring an optimum balance between the floor slab and column design. Shorter, wider columns are typically more economical as column elements, while longer thinner columns typically enable a more economical floor slab design while fitting more unobtrusively within architectural layouts. Consideration needs to be given to additional, secondary forces attracted due to the wind and seismic actions when using longer columns, however, and a study shall be undertaken to derive the most cost-effective overall arrangement of columns and slab. Detailed coordination in this regard is needed from the scheme design stage.

Typically, column lengths shall remain uniform through the height of the building as far as practical to ensure a consistent, economical floor slab design, while changes in column size to suit reducing loads shall generally be made by adjusting the column width. At the point where column widths cannot reduce further due to limiting code or construction issues (typically 250 to 300mm), the concrete grade shall be adjusted accordingly to suit load demand. See Section 5 for material specifications for preferred concrete grades.

Column design should generally be based on 1% reinforcement minimum as designed by code, though local increases as required for practical considerations shall be permitted.

As far as practical, columns shall not ordinarily be assumed to participate in resisting lateral loads unless their length dictates otherwise.

Wall design should generally be based on 0.25% minimum reinforcement as designed by code.

For initial design in columns, below the recommendation lead to practical design and reduce the time frame of iteration of the design.

- Minimum column dimensions for 'stocky', non-sway braced frame is = clear height/15
- Minimum column dimensions for the unbraced frame is = clear height/10

4.d.1.3 Lateral load resisting structural system

As discussed in the previous section on the vertical load-resisting system, a central concrete core is typically anticipated as part of the vertical system, and it is expected that this shall also form the main component of the lateral load resisting structural system, typically the only component.

Typically in a tall building, the core dimension is dictated by the efficient arrangement of elevators, stairs, MEP rooms, communication rooms, and vertical risers, as well as the depth of the apartments from perimeter to circulation space. The resulting cellular core can typically provide most, if not all, of the stiffness required by the building to resist lateral forces and to limit lateral deflections during wind and seismic events.

In order to maximize structural efficiency of the core, the outer walls should be as continuous as possible (i.e. as few openings as possible) and internal walls parallel with the narrower aspect of the floor plate should act as continuous links between the outer walls as far as possible, again with minimum openings to carry the shear in that direction. It is expected that inner walls shall be kept as thin as possible, primarily sized for gravity and shear forces, with outer walls being thicker, sized to provide required stiffness of the overall core and to satisfy strength requirements.

Singular disconnected shear walls are inefficient in comparison to a well-considered core in resisting lateral forces, and shall generally be avoided where possible.

Emaar's preferred approach for the purpose of this guideline document is a tall building with a single central, or near the central, well-connected core, as opposed to several smaller elements of core or other disconnected singular walls. Centrally located cores reduce torsional forces on the core under wind and earthquake actions resulting in more economical wall thicknesses.

Please note that for rectangular floor plates of high aspect ratio, a mere central core may not be sufficient or efficient and additional shear walls or lateral systems may be necessary. Such buildings are beyond the scope of this document.

In some situations, the requirements of the architectural floor plan may suggest an eccentric core placement such as shown in the adjacent figure.

Such an eccentric core will attract torsional forces which may necessitate additional thickening of the core walls and/or supplementary stiffness through a second LLRSS such as moment-resisting frames located at the periphery of the floor plate, which is inefficient.

An eccentric core is discouraged, and approval to proceed with such systems shall be sought from Emaar before proceeding.

Alternative systems, including moment-resisting frames and outrigger systems, may be considered in special circumstances with prior approval from Emaar, but are generally not considered appropriate or efficient for the typical buildings covered by this guide.

4.d.1.4 Load transfers

Transfer structures are elements used to deviate vertical or lateral load paths over one or more levels of a building. These are typically used where a change of use on a floor dictates a different column or wall arrangement, or to accommodate architectural features.

Transfer structures are structurally inefficient and typically have a significant design, cost, and construction schedule implications, requiring careful consideration of construction logistics, as well as consideration of the impact of long term deflections of the transfer members and supporting elements. As such, transfer structures should be avoided where possible.

In particular, the transfer of elements of the lateral load resisting structural system should be avoided, as such transfers add risk, and the resulting inefficiencies in the design are amplified by code requirements for magnified design forces.
Any transfers should be agreed in advance with Emaar, but where necessary, generally limited to the upper levels of tall buildings where forces being transferred are small in nature.

4.d.1.5 Foundations

For the type of structure under consideration, it is expected that the foundation will generally be piled for buildings with no basement or raft on pile system where basement(s) are planned. A good quality geotechnical site investigation, including high-quality sampling procedures and a sound engineering interpretation of results, will go a long way in arriving at an economical foundation system. Reference should be made to Section 7 for the specification of the site investigation, and it is highly recommended that a geotechnical consultant be approached early on in the design process to assist in the production of an economical design solution.

The economical design of foundation systems requires an iterative process between structural designer and geotechnical consultant. The design data from the structural analysis is used to determine geotechnical design parameters such as vertical and horizontal stiffness of the foundation, and this, in turn, alters the behavior of the structure. The compatibility between geotechnical analysis and structural design is crucial so that a converged foundation settlement is reached within a reasonable tolerance.

Attention must be paid to the design and or construction sequencing of rafts for the effects of the heat of hydration, long term temperature variation, shrinkag, and restraints effect due to the presence of piles retaining walls and other vertical structure. A single raft pour is generally preferred, particularly for foundations below the natural water table, to ensure reduced risk of water ingress at construction joints.

Dewatering costs are appreciable in any project with a basement below the natural groundwater level. The consultant should determine the stage of completion of the structure at which dewatering can be safely turned off to minimize this duration. Advance excavation and foundation packages should also consider the overall construction schedule so that excavations are not left open for extended durations between completion of early enabling or foundation works, and commencement of main structural works.

Following are some conditions that need to be accounted for in the design of foundation systems:

1. Structural design at ULS conditions.
2. Relevant SLS conditions, including crack widths on the internal and external face.
3. Temporary conditions, including temporary uplift due to water pressure when dewatering will be turned off.
4. Sensitivity analysis of main design parameters, such as the effect of variation of pile stiffness.
5. Short term temperature change due to heat of hydration and shrinkage.
6. Effect of restraint offered by pile head to raft movement due to temperature forces.
7. Effect on foundation due to dewatering in the adjacent plot during the design life of the building

4.d.2 Key Design Consideration

4.d.2.1 Wind Design Requirements

Wind loads on building structures are both static and dynamic in nature but are typically applied as equivalent static loads. The actual wind load is dependent upon the shape, height and dynamic characteristics of the building, the building's surroundings and orientation, and the local wind climate.

Dubai Municipality has defined the design wind speed as a 3-second wind gust speed of 38 ms-1 at a reference height of 10m above ground, and this speed should be used for design unless otherwise agreed with DM and Emaar. This wind speed has a return period of 50 years and is compatible with ASCE 7-05.

Dubai Municipality allows for the derivation of wind from either code-based methods, or from a wind tunnel study carried out by a competent wind tunnel laboratory that models the building's shape and dynamic characteristics, as well as the site location, local surroundings, and local wind climate.
Due to the more detailed consideration of in-situ conditions, and to the use of statistically derived directionality of local wind conditions, it is expected that the use of wind tunnel results will result in a more economical structure, and it is thus mandatory that a wind tunnel study is undertaken on all tall buildings reaching 120 m in height or more. Depends on the configuration of surrounded buildings around of project, the wind tunnel should cover all possible scenarios and configurations (Existing and Future) to obtain govern loading condition.

The following section further discusses considerations for wind tunnel testing.

4.d.2.2 Considerations for Wind Tunnel Testing

Wind tunnel testing is a method of estimating wind loads on tall building structures and their cladding by considering wind-structure interaction on a physical model of the building and its surrounding in a wind tunnel laboratory.

If the significant impact on the building is expected due to wind loading, early wind tunnel results should be considered for study during the concept design phase, otherwise, the wind tunnel analysis is expected to be undertaken once the building's envelope is effectively frozen and no significant changes are expected.

The wind tunnel laboratory is to be responsible for the execution of the model testing including the derivation of local wind climate, modeling of appropriate incident wind characteristics in the wind tunnel, model making, instrumentation, measurements and evaluation of the results. The consultant shall at all times review and interrogate the results provided and comment as appropriate.

The wind tunnel laboratory is required to provide a method statement suitable for submission to local regulatory authorities for comments and approval and shall ensure that the testing methods and wind parameters meet all current local requirements.

The wind tunnel laboratory report is to contain details of the experimental and analysis techniques and quality procedures used to ensure correct calibration of instrumentation in addition to the presentation of the measured values and discussion of the use of the measurements.

Following studies are generally carried out in wind tunnel analysis for structural purposes:

- Wind climate analysis and derivation of data for the proposed site.
- A High-Frequency Force Balance test (HFFB) is typically expected to be carried out to provide the global base forces (shear and overturning moments) for design.

Other methods, including High-Frequency Pressure Integration (HFPI) testing and aero-elastic testing, are typically not appropriate for buildings targeted by these guidelines.

Wind tunnel testing shall be in accordance with international best practice for wind tunnel testing, and reference should be made to Wind Tunnel Testing of High Rise Buildings, a Technical Guide published by the Council on Tall Buildings and Urban Habitat, and should meet or exceed the requirements of ASCE 7-05 and the ASCE Manual of Practice No. 67 for Wind Tunnel Testing of Buildings and Structures. The studies shall also meet the requirements of the Dubai Municipality with respect to required codes of practice and specified wind parameters.

4.d.2.2.1 Input from Structural Designer

The structural designer shall typically provide structural data for the building including parameters such as:

- Building's geometrical data.
- Floor by floor diaphragm mass data)
- Mode shapes
- Modal periods and frequencies
- Inherent damping value(s) should be specified by the consultant structural engineer

4.d.2.2.2 Technical requirements

A. Preliminary Desktop Study

At project commencement, it will be decided if a desktop study should be carried out to provide indicative pressures for both the structural and façade cladding loading, making due consideration of the building shape.

B. Advance Testing Structural Loading

Where appropriate, a HFFB test shall be carried out at commencement to provide advance information for global structural forces. It is expected that the HFFB test base shall be positioned at the intersection of the main building with the podium, with a separate allowance provided for the podium component of the load.

C. Building and Proximity Models

The building model(s) and surroundings (proximity model) shall generally be constructed at a scale of approximately 1:250 to 1:500, with the final scale to be mutually agreed. The construction and suitability of these models for the appropriate wind tunnel measurements are the responsibility of the wind tunnel laboratory but shall consider such wind characteristics as Reynold's number and the sensitivity of the measuring equipment. Details of the model construction, wind speed measurement, and pressure tap locations, etc., are to be discussed and agreed with the Consultant before model construction.

The proximity model is to include general massing of the buildings and street patterns to a radius of at least 500m from the site with more detail for buildings within very close proximity. Two proximity models, one including current conditions and one including any known future construction within the vicinity must be modeled, and the most onerous of the two conditions considered for design.

The model shall include detail of the upper parts of all tall buildings within the 500m radius which have no intervening tall buildings between them and the site.

Photographs of the building and proximity model are to be provided in the report. Sketches of the model showing principal dimensions are also to be included in the report.

D. Wind Climate

Wind climate data to be used by the wind laboratory are to be provided to the Consultant for review and further use in conjunction with the project.

It shall be ensured that any historical wind speed and directionality characteristics are obtained from reliable anemometers with appropriate response characteristics. Corrections for siting and shielding effects shall be applied if required, and these shall be fully described. The analysis shall be conducted using a minimum of 10 years of data, at the same time taking into consideration the building life cycle of 50 years. The climate study shall also include local weather phenomena such as Shamal storms.

Design wind speeds shall, where necessary, be scaled to meet local regulatory requirements

E. High-Frequency Force Balance Studies

The high-frequency force balance technique provides provide convenient measurements of integrated forces on rigid building models for base moments, torque and shear force. The test model is mounted on a balance consisting of a rigid frame with force links and miniature load cells capable of measuring six components. The measurement of forces obtained is then distributed along with the height of the structure in a manner where for both overall base shear and overturning moment are satisfied.

A full specification and method statement shall be provided by the wind tunnel laboratory for the HFFB testing for review and approval before commencement.

F. Cladding Pressure Studies

For cladding pressure studies for the design of façade systems, it is anticipated that a pressure tap model shall be constructed and measurements were taken for at least 36 wind directions. The average pressure tap density is expected to be not less than one tap per 3 m^2 of building surface under investigation, or as otherwise mutually agreed.

For canopies and walls open to the air on both sides, the differential pressures are to be measured by direct comparison of pressures from pressure taps on opposite faces.

Pressure measurements are to be given in terms of mean and maximum and minimum pressures determined by extreme value analysis of 1-second (full-scale) averages. Time histories of the pressure measurements shall be stored and provided to Consultant on request.

Sensitivity information shall be provided for changes to surface roughness.

Further reference shall be made to the façade section of this guide.

G. Other Studies

Other studies, including environmental wind studies and specific MEP studies for air intake and exhaust, etc. may be appropriate for the design of the assessment of wind comfort in and around the building, and design of MEP services. In such cases, the consultant shall liaise with other consultants so that a consolidated specification for the wind tunnel testing can be prepared.

4.d.2.2.3 Report

The report is to include a full description of the measurement, analysis and quality assurance procedures and any difficulties with any of the measurements; three copies of each report shall be supplied

Structural loads shall be provided as equivalent static floor-by-floor forces and shall include sensitivity studies on building mass and dynamic characteristics. Appropriate joint action forces for combining Fx, Fy and Mz shall be recommended.

Cladding pressure loads shall be presented as equivalent static loads, and shall be reported using contoured, recommended design pressure diagrams plotted on elevations and plans of the buildings (with soft copy 3D models and excel format). Diagrams showing pressure measurement locations and peak-measured pressures at each location shall also be supplied. Appropriate load combinations shall be recommended.

Videos and photographs shall be supplied of all models and test procedures as appropriate. Photographs of the completed models shall be supplied for review by the design team prior to the commencement of testing.

All tabular values and spectra are to be provided electronically in a format to be agreed with the Consultant.

The wind studies report shall satisfy the requirements of the CTBUH guide, and the ANSI/ASCE Standards "Minimum Design Loads for Buildings and Other Structures".

Any dynamic parameters provided by the consultant for the purposes of wind tunnel testing shall also be quoted in the report.

4.d.2.2.4 Other considerations

Building Deflection and Drift

Some codes limit wind movement in terms of maximum building deflection and inter-story drift ratios, and others leave this more to the engineering judgment of the designer, where the sensitivity of elevators and building cladding systems, etc. to movement guide the basis of the limiting values. Drift is calculated by the structural engineer based upon effective wind forces and combinations provided in the wind tunnel report, and proposed limits are provided later in this guide.

Movement and drift are typically calculated for a lower wind return period of 10 years and separate wind forces for the deflection and drift cases shall be provided by the wind consultant.

Top floor acceleration and Torsional velocity

Resonant components of wind turbulence generate accelerations and torsional velocities in tall buildings, typically most notable at higher floors.

The wind tunnel testing shall determine expected accelerations and torsional velocities for a range of return periods and reviewed against acceptable criteria provided later in this guide

4.d.2.3 Seismic Design Requirements

The United Arab Emirates and Dubai individually lie within the northeastern part of the Arabian Peninsula, southwest of Asia. Generally, the UAE is known for its low seismic activity despite being surrounded by major faults and other seismic sources. Until quite recently, seismic hazard in the UAE was considered to be negligible. However, due to the rapid development of high-rise construction in the UAE, significant attention has been drawn to the risk posed by earthquakes to the buildings and infrastructure of the country. In addition, the country is prone to moderate seismic activity and to the impact of strong earthquakes originating from Iran. One of the main sources for this far of fault event is located in Zagros belt focal mechanism adjacent to Iran. The main As a result, various efforts emerged that aimed at assessing the seismic hazard of the UAE; and while these efforts reported varying results, most of the studies concluded that the UAE is susceptible to moderate seismic hazard. Following above statement, forces used for seismic design of tall buildings must conform with the requirements of Dubai Municipality as provided by DM Circular no. 37.

Seismic forces are generated in the building as inertial forces due to the movement of the ground beneath them, and the building dynamically responds to the ground acceleration as the ground shakes. The magnitude of forces differs in each structure due to the following factors:

1. Structural properties
    a. Mass of the building and the distribution along the building height
    b. Height of the structure
    c. The arrangement, energy dissipative mechanism, and stiffness of the LLRSS

2. Site geology and seismicity
    a. The soil profile of the site
    b. Proximity to a fault and other seismic sources
    c. Recorded ground shaking/acceleration e.g. Peak Ground Acceleration (PGA)

Building codes allow for a simplified way of representing the complex, dynamic, seismic forces provided that the structure meets certain requirements. Inapplicable cases, the inertial forces can be represented by a series of static loads applied at each floor. Linear Response Spectrum Analysis (LRSA) is a recommended method for a normal structure which is mostly covered by this guideline. In some cases, more complex analysis including of Linear Time History Analysis (LTHA) is required, but would not be expected to be considered for the structure covered by this guide.

The performance-based design (PBD) is not covered in this guideline and just performance objective indication listed for further study in case of consultant requirements. For PBD, Applicable documents and codes such as TBI, LATBSD, ASCE41, and FEMA should be considered.

There are several accepted ways to determined equivalent static forces for design as specified by design codes, but the most appropriate method which is recommended for tall buildings governed by this guide is that of an elastic response spectrum analysis. Tall buildings governed by this guide should follow this method where permitted by code to ensure a more appropriate distribution of force, which is typically expected to lead to a more economical structural system.

Building design codes set a level of risk for which the structure shall be designed. The most common approach is to consider a level of ground shaking for a return period of 475 years, representing an event with a probability of exceedance of 10% in 50 years. This level of ground shaking is referred to as the Design Basis Earthquake (DBE) and structures designed to meet this level of ground shaking are said to satisfy a Life Safety level of performance.

Earthquake Having Probability of Exceedance	**Mean Return Period (years)**
50% / 50yr	72
20%/50yr	225
10%/50yr (DBE)	475
2%/50yr (MCE_R)	2475

4.d.2.3.1 Governing Seismic Code

UBC 97 used to be the most followed seismic code for the buildings in Dubai. Dubai municipality released circular no. 37 specifying the use of ASCE 7 for buildings.

DM Circular no. 37 specifies the ground movement parameters to be followed, these are provided as well in section 4.f.3.

4.d.2.3.2 Seismic mass

Noted as one of the factors affecting seismic loads, the seismic mass to be considered for design shall be carefully determined. Effective mass shall be in accordance with ASCE 7. A minimum of 25% of the design live load shall also be included in the seismic mass. The load from MEP installations sometimes categorized as Live Load, shall be fully included in the seismic mass.

4.d.2.3.3 Earthquake loads for ULS design

The basic seismic forces to be used for design shall include the two components ρEh and Ev for vertical and horizontal components.

Where required by ASCE 7, some elements of the building, such as transfers structures as discussed earlier, may need to be designed using the estimated maximum earthquake force, Em. This ensures that critical elements that may experience excessive deformations will remain elastic during a seismic event.

The magnitude of force generated in the structure due to seismic ground motion is a function of the energy dissipation characteristics of the lateral load-resisting structural system (LLRSS). Building codes assign a ductility factor, R to represent this characteristic for different forms of LLRSS'.

Buildings designed to conform to these guidelines are expected to be classified as bearing wall systems with concrete shear walls (R factor as per ASCE 7). The boundary between a bearing wall system and building frame system can be a point of debate between Engineers, but a bearing wall system is classified as one where a significant portion of floor load is carried by the walls. This is a common arrangement in tall buildings and is the system proposed by this guide.

4.d.2.3.4 Preferred method of seismic analysis

As discussed earlier, it is preferable to use the response spectrum method for the design of building structures, as this method captures the building's structural behavior more realistically than more simplified static procedures, and better predicts the distribution of earthquake forces along with the height of the building. The dynamic response of typical tall buildings considered in the development of these Guidelines includes first translational mode mass participation in the range of 40% to 70% of the total, with approximately 90% mass participation represented in each principal direction of response when the first three to four translational modes are considered.. Given the time periods, spectral accelerations for each mode can be found from the response spectrum, and further calculations using the participating mass, spectral accelerations, and mode shapes are then used to calculate the member forces, base shear, etc. as outlined in the design code.

An overall economy is expected in the building structure through the use of a response spectrum analysis, and it is the only permitted choice if the building contains any of the structural irregularities stipulated in ASCE 7 or applicable code. Further reduction may be achieved through the use of a site-specific elastic design response spectrum, but it is not expected that this will be pursued on projects considered under these guidelines. Any use of this approach shall be discussed and agreed in advance with Dubai municipality.

When assessing design forces using the elastic response spectrum, care should be taken when using the results of response spectrum analysis combinations for foundation design. These results are often given in scalar values losing the sense and direction of the reactions. The Consultant shall take necessary steps to account for the proper sign conventions for the foundation loads and capture overturning effects of the core wall. Use of static seismic loads may be more sensible to use for foundation, however, the Consultant shall ensure that these static forces similarly capture the peak shear and overturning moment observed from the response spectrum analysis.

4.d.2.3.5 Orthogonal Effects

Structures in Dubai will need to be designed for earthquake forces acting in a direction other than the principal axes if found applicable as per ASCE 7. Reference should be made to code provisions for dealing with such orthogonal combinations.

4.d.2.3.6 Performance Objective level based

Buildings designed in accordance with these Guidelines are intended to have seismic performance capability at least equal to, and in some respects superior to, that intended for similar buildings designed in full conformance with the prescriptive requirements of ASCE 7. As presented in the Commentary to FEMA P1050 (2015), most of the tall building and normal building fit into the risk category II, the building code is intended to provide Risk Category II buildings the capability to:

- Withstand Risk-targeted Maximum Considered Earthquake (MCER) shaking, with low probability (not more than 10%) of either total or partial collapse;
- Withstand Design Earthquake (DBE) shaking, having an intensity two-thirds that of MCER shaking, without generation of significant hazards to individual lives such as falling debris or nonstructural components; and
- Withstand relatively frequent, more moderate-intensity earthquake shaking with limited damage.

4.d.2.3.7 Self-straining Forces

The structure shall be designed to resist any self-straining forces arising from the restraint of contraction or expansion of structural elements resulting from temperature change, shrinkage, moisture change, creep in component materials, movement due to differential settlement, or combinations thereof.

In the first instance, appropriate detailing should be provided to eliminate such stresses, but where such detailing is not practical, or cannot fully alleviate these stresses, analysis of their effects shall be performed and designed for. CIRIA Guide C 660 is recommended as a good guide for providing suitable assumptions and design parameters, as well as ICE Manual for Structural Design, Chapter 10.

As an over-arching guide, reasonable assumptions should be made regarding parameters not specified by code, such as allowance for cracking etc. to relieve restraint, without undue conservatism.

4.d.2.3.8 Long-term Thermal Forces

For long-term thermal effects on concrete elements, a temperature range of +/- 25^{0}C, is recommended. This is based on the guidance of CIRIA C660 which suggests the use of mean monthly temperatures to establish an appropriate range, and on available data from the National Centre for Meteorology and Seismology, and the Department of Civil Aviation – Dubai, as shown in the following Figures. Mean monthly figures are selected due to the rate of response to change in temperature of concrete. For steel elements, and for elements directly exposed to solar radiation, a higher range may be appropriate (refer to CIRIA C660).

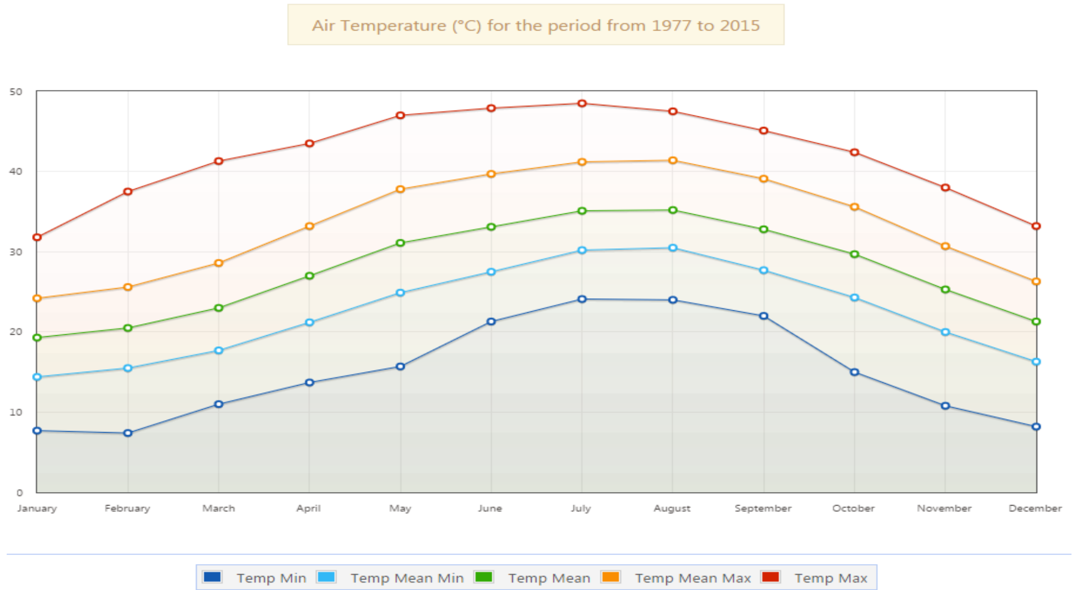


Figure 4-1 Data from National Centre for Meteorology and Seismology (http://www.ncms.ae/). (Refer to Green Curve).

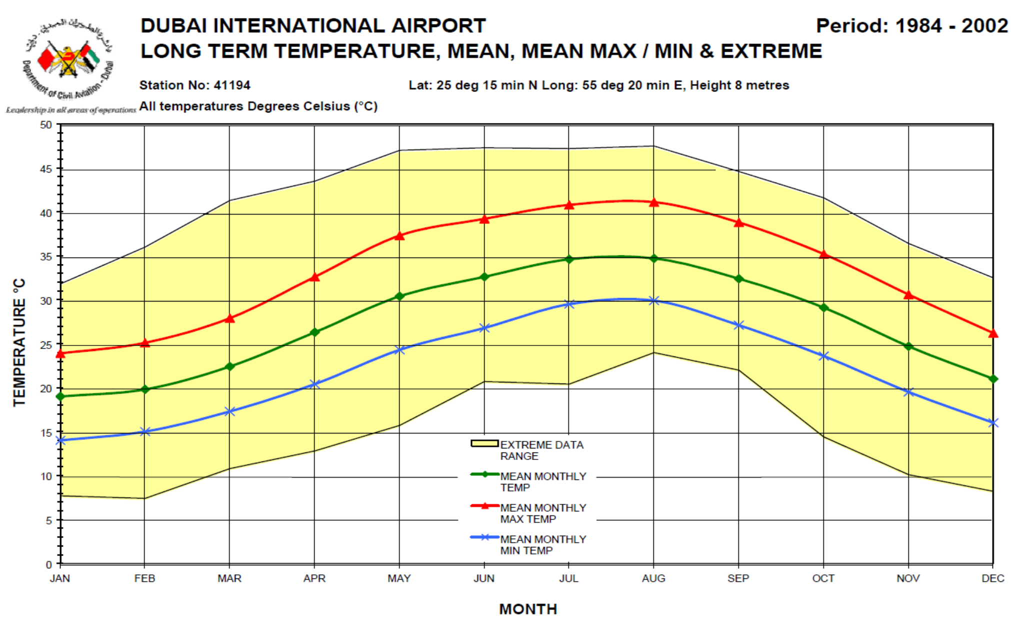


Figure 4-2 Long-Term Climatological Data from Department of Civil Aviation – Dubai. (Refer to Green Curve).

From both sets of mean monthly data, and more detailed data related to the DCA records, maximum and minimum monthly means of 36.5^{o}C and 17.4^{o}C are observed. For simplicity, these figures have been rounded to 40^{o}C and 15^{o}C respectively to provide the 25^{o}C range specified.

4.d.2.3.9 Shrinkage and Early Thermal Forces

Reference should be made to CIRIA C660 for assessing for the effects of early thermal and long-term shrinkage effects. Care must be taken to allow for relief through cracking of structural elements and a reasonable representation of restraint to ensure that resultant designs are not overly conservative.

Cracks resulting from the restraint of movement should be assessed against acceptable limits.

4.d.2.3.10 Axial Shortening and Differential Movement

The presence of sustained axial loads in columns and core walls causes axial shortening of those elements. All elements are subject to short-term elastic shortening under compressive loads, but in reinforced concrete structures, axial shortening is further amplified due to shrinkage and creep of concrete. As column elements are generally designed considering efficiency in carrying these gravity loads, while the central core is designed principally for resisting lateral loads, there is often a difference in the strain to which the columns and core walls are subjected, which leads to differential movement. This difference in axial shortening can cause additional moment and shear in the slabs and beams connecting core walls to peripheral columns, most notable where the distance between the elements is relatively short.

All buildings should carry out representative studies to assess expected differential movement of vertical structural elements to define any possible issues, while more detailed studies will be required where the movement from initial, brief studies show that significant differential is possible.

The expected sequence of construction shall be considered in the analysis.

4.d.2.3.11 Coupling beam design

The presence of coupling beams and its related design requirements can be clarified as per the below:

- Bearing wall system (R = 4.5) – SDC C – then walls and link beams are designed as ordinary
- Coupled shear walls system (R= 5.5) – SDC C - with coupling beams then these beams are to be designed as IMRF and the walls as Ordinary shear walls. If there are frames outside this coupled shear wall system then these can be designed as ordinary frames.

4.d.3 Construction Considerations

4.d.3.1 Internal partitions on residential floors

The conventional construction of partitions walls on the residential floors can add up significant dead weight resulting in increased sizes of floor slab, columns, core walls and foundation. It is recommended that efforts shall be made to use light-weight materials in the construction of partition walls.

Three forms of partition walls need to be considered:

- Walls separating apartments from other apartments and common areas.
- Walls around wet areas within apartments.
- Other internal partition walls.

Partition build-ups need to be coordinated with the architect, but the following outlines Emaar's expectations for each type of partition. The main purpose of the following requirements is to reduce weight on the structure, and deviation from this needs to be agreed in advance with Emaar.

Walls Separating Apartments and Common Areas.

Such walls often called as demising walls are required to provide required acoustical separation between apartments and shall consist of 200 thick AAC blocks with 12mm thick plater board on both faces.

Walls Enclosing Wet Areas

Walls around wet areas need to be capable of carrying the heavier wall finishes associated with wet areas and carry waterproofing membrane. It is expected that these walls will consist of light weight AAC blocks 100 mm thick with tiled finish on internal face and 12mm thick plasterboard on the outer face.

Other Internal Partitions

Lightweight drywall partitions shall be considered for all other partition walls within apartments. The build-up is expected to consist of 100 mm thick dry wall partition consisting of 12mm thick plasterboard on each face attached to light gauge steel framework. It is expected that these will have insulation for Acoustic separation.

Refer to section 6.1.5 for partition wall loads to be used in design and Appendix-A outlining suggested procedure for calculating the partition walls loads on the slab panel.

4.d.3.2 Floor finish

It is recommended that the thickness of the build-up floor finishes for mortar bed for floor tile installation, shall be minimised in order to reduce the superimposed dead load. The reduction in finish thickness will affect the design of slabs and more importantly the cumulative effect over many floors can result in appreciable reduction in column loads and foundation design. The reduction in mortar bed depth can be achieved by trowel finishing the slab surface to a reasonably plane surface, with an overall finishes thickness of 70mm deemed appropriate.

The washroom floor is typically required to have some slope and therefore the detail of the falls within the wet area and the interface between room finish and wet area finish is critical in achieving the reduced mortar bed.

4.d.3.3 Waterproofing

The external underground waterproofing membrane not the first line of defence against seepage of water into the building but also protects the concrete from harmful chlorides and sulphates present in the soils of the region. The aim of the consultant shall not be to solely rely on waterproofing membrane presence but all substructure reinforced concrete elements shall be designed to limit the crack-width under service loads; Dubai municipality specifies a limiting crack-width of 0.2mm for concrete external faces. The reinforced concrete underground elements shall be designed for the effect of superimposed loads, water and soil pressure, and stresses due to temperature and shrinkage effects, all combined in accordance with ASCE 7-05. The designer shall also pay attention that the layout of the raft and retaining wall affords the easy and convenient installation of the waterproofing membrane. The effort shall be made to avoid sharp corners, sharp edges, and protrusions which make reliable installation of the membrane a difficult task.

Special attention must be paid to construction, contraction and expansion joints. Allowance should be made for future remediation of the waterproofing system should future leakage occur, such as re-injectable systems, etc..

Refer to section 6.9.1 for applicable crack width limits.

4.d.3.4 Floor to Floor Height

The preferred typical floor to floor height for residential building is set at 3.3m, with the requirement to provide 2.5m clear height in the corridors and 2.9m clear inside the apartment's living and bedrooms. In order to provide maximum clear ceiling height, no false ceiling is preferred in living areas and bedrooms. The MEP and other special floors will have greater floor to floor heights as per the requirements of MEP design.

The floor to floor height in the parking area is kept at 3.2m unless a greater clearance is required for areas accessible to service vehicles.

Floor to floor heights should be fully coordinated among all parties at the concept stage and agreed with EMAAR in case of deviation.

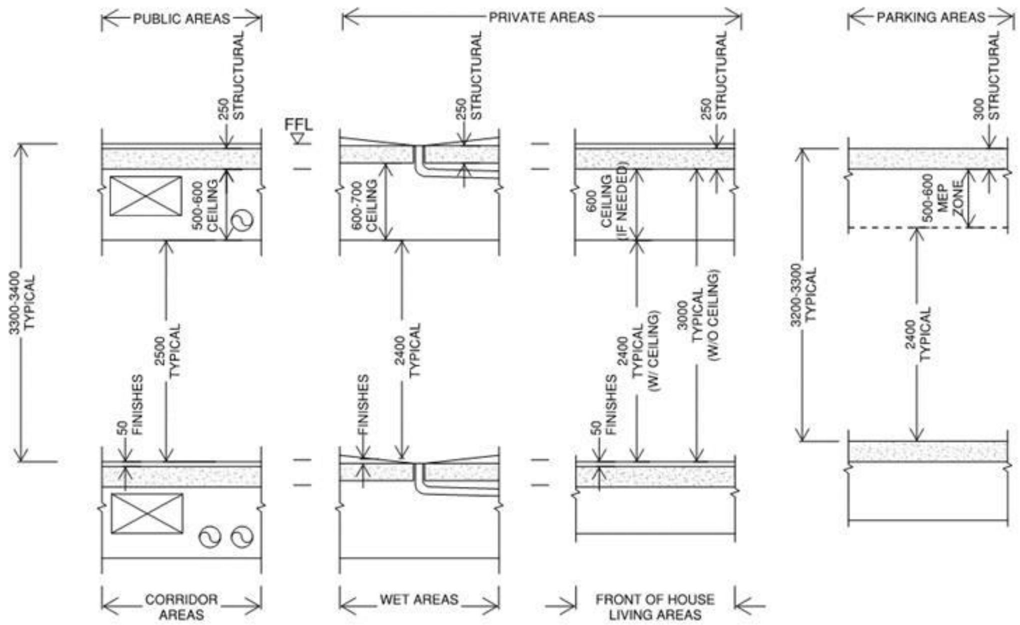


Figure 4-3 Recommended Floor to Floor Heights

4.e Materials

The materials listed in this section are the preferred materials/grades/sizes for design purposes. See the Drawings and Specifications for the project requirements for construction.

4.e.1 Concrete

4.e.1.1 Concrete Grades

The preferred concrete grades and locations in tower elements are as follows:

Locations	*fc' (MPa)
Foundations & perimeter Retaining Walls3	40
Slabs and Stairs (including gravity beams within the slab)	40 to 50 (or higher if beneficial in reduction of PT, rebar, etc)
Columns.	40 for upper levels up to 75 for lower levels
Shear Walls (and Coupling Beams)	40 for upper levels up to 75 for lower levels

* f'c = 28-day cylinder compressive strength, however it is allowed to achieve the specified concrete cylinder strength of 65 to 75 MPa at 56 days

For podium structure, not supporting tower elements, the following concrete grades are preferable:

Locations	*fc' (MPa)
Foundations & perimeter Retaining Walls[3]	40
Slabs and Stairs (including gravity beams within the slab)	40 to 50 (or higher if beneficial in reduction of PT, rebar, etc)
Columns.	40 to 50
Shear Walls (and Coupling Beams)	40 to 50

* f'c = 28-day cylinder compressive strength

Note:

1. For blinding and screed, 20.0 MPa can be considered, lower grades may be acceptable depending on the aggressiveness of the contact soil.
2. The ratio of column/shear wall concrete strength to slab column strength must not exceed 1.4 to avoid requirement for puddling procedure. ACI 318M-11, section 10.12. Alternatively, such differential may be permitted where actual forces carried by columns do not exceed this ratio.
3. For structures subject to severe environmental exposure, minimum concrete strength may also be driven by durability requirements

Concrete grades and locations are as follows for ancillary structures:

Locations	*fc' (MPa)
Foundations & perimeter Retaining Walls3	40
Slabs and Stairs (including gravity beams within the slab)	40
Columns.	40
Shear Walls (and Coupling Beams)	40

* f'c = 28-day cylinder compressive strength

4.e.1.2 Other concrete properties

For the purpose of analysis, additional concrete properties are given below.

Property	**Value**	**Notes**
Density	24.5 kN/m3 18 kN/m3	Normal-weight concrete including allowance for reinforcement Light-weight concrete including allowance for reinforcement
Elastic modulus	0.043 x wc1.5 x sqrt (f'c)	ACI 318M-14 cl 19.2.2.1.a
Poisson's Ratio	0.2	
Coefficient of Thermal Expansion	10 x 10-6 /ºC	Value depends upon the source of the aggregate used. Thermal coefficient is lowest for lime stone and highest for quartzite with a value ranging from 7.5x10-6 to 12x10-6 / ºC. The value shown in this table is commonly used in the region (granite).

4.e.2 Reinforcement

Deformed reinforcing bars:

Grade 460 (fy = 460 Mpa for main/flexural rebars)

Grade 460 with 420 MPa for shear rebars

Note: Reinforcing bars grade 500 MPa can be used as well after agreement with Emaar and Authorities.

Welded wire fabric (deformed): ASTM A1064M (fy = 480 MPa)

- The Approximate diameter to outside deformations should be used only for detailing locations with congested rebar. Source CRSI Detailing Manual page C-10 and ACI Detailing manual Supporting reference data page 195.

- Use of T40 reinforcement shall be limited, and where used, couplers shall be used at lap locations

Bar sizes	Nominal Dimensions (For Design)	
	Diameter (mm)	Area (mm2)
T8	8	50.3
T10	10	78.5
T12	12	113
T16	16	201
T20	20	314
T25	25	491
T32	32	804
T40	40	1256

Note: For detailing, appropriate rebar diameters to be used.

4.e.3 Structural Steel

4.e.3.1 Steel Grades

The Tables below summarize the material grades for structural steel that can be used in EMAAR projects, unless otherwise noted. Value indicated are subject to market availability.

For American steel sections the following materials may be used.

Type of Section	Material Specification	Fy MPa (ksi)	Fu MPa (ksi)	Select shapes from
Wide Flange	ASTM A992M Grade 345	345 (50)	450 (65)	AISC Manual
Channels	ASTM A36M	250 (36)	400 (58)	AISC Manual
Angles	ASTM A36M	250 (36)	400 (58)	AISC Manual
Plates	ASTM A36M	250 (36)	400 (58)	See AISC Manual for standard thicknesses
Built-up sections	ASTM A36M	345 (50)	450 (65)	
HSS Rectangular or Square	ASTM A500 Grade B	240 (35)	400 (58)	AISC Manual
HSS Round	ASTM A500 Grade B	240 (35)	400 (58)	AISC Manual
Pipe	A501	240 (35)	400 (58)	AISC Manual, ASTM specs

Type of Section	Material Specification	Fy MPa (ksi)	Fu MPa (ksi)	Select shapes from
Pipe	A53M, Grade	240 (35)	415 (60)	AISC Manual, ASTM specs
(alternate)	B Type E or S			
WT	ASTM A992M	345 (50)	450 (65)	AISC Manual

Likewise, for British standard sizes, the following material specifications may be used.

Type of Section	Material Specification	Fy MPa	Fu MPa	Select shapes from
Beam	BS EN 10025 Grade 355J0 or Grade 275J0	295 to 355 (based on thickness) 225 to 275 (based on thickness)	490 (for thickness less than 100mm) 410 (for thickness less than 100mm)	Steelwork Design Guide to BS 5950-1: 2000 By SCI
Universal Column	BS EN 10025 Grade 355J0 or Grade 275J0	295 to 355 (based on thickness) 225 to 275 (based on thickness)	490 (for thickness less than 100mm) 410 (for thickness less than 100mm)	Steelwork Design Guide to BS 5950-1: 2000 By SCI
Angles	BS EB 10056-2 Grade S275	275	410	Steelwork Design Guide to BS 5950-1: 2000 By SCI
Bars and Plates	BS EN 10025 Grade S275 or Grade S355	295 to 355 (based on thickness) 225 to 275 (based on thickness)	490 (for thickness less than 100mm) 410 (for thickness less than 100mm)	Refer to market available sizes
Hot-finished SHS, RHS and CHS	BS EN 10210 Grade S355J2H	240 (35)	400 (58)	Steelwork Design Guide to BS 5950-1: 2000 By SCI
Cold-formed SHS, RHS and RHS	BS EN 10219 Grade S355J2H	240 (35)	400 (58)	Steelwork Design Guide to BS 5950-1: 2000 By SCI

Note that among the two material specifications given in the preceding tables it is preferred to use the British standard sections as it is more commonly used in the UAE. For British sections, grade S355 is typically used when strength governs the design while grade S275 is used when stiffness drives the design. For secondary steel structures, grade S275 is typically used.

4.e.3.2 Steel Properties

Density:	7850 Kg/m^3
Young's Modulus:	E=200,000 MPa
Poisson's Ratio:	ν = 0.30
Coefficient of thermal expansion:	α = 11.7x10-6/oC

4.e.3.3 High-strength Structural Steel

Apart from the steel grades mentioned in above tables, high-strength structural steel can be used as well in Emaar portfolio. The acceptable steel grades are HISTAR 355 and HISTAR 460 or above.

Due consideration needs to be given to reduction of yield strength as a function of the thickness.

4.e.4 Bolts

4.e.4.1 Bolt Materials

For structural steel connections: ASTM A325M

Preferred sizes: M20 [3/4"Φ], M24 [1" Φ]

fy = 660 MPa [96 ksi]

fu = 830 MPa [120 ksi]

For non-structural steel connections: ASTM A307

fy = 414 MPa [60 ksi]

4.e.4.2 Bolt Tensioning

All bolts are bearing bolts (N - threads included) unless specifically noted as slip-critical (SC).

All bolted connections must be designed and constructed per the requirements of AISC "Manual of Steel Construction" including AISC "Specification for Structural Joints Using ASTM A325 or A490 Bolts". In addition to these requirements the following requirements must also be met:

1. Slip-Critical Connections: Slip-Critical bolts shall be fully tensioned with faying surface preparation

   Class A. Slip-Critical connections shall be used in the following locations:

   a. Wherever required in the AISC Provisions
   b. Wherever noted on the drawings
   c. Whenever oversized holes are used.
   d. At all connections for members directly or indirectly supporting mechanical equipment and stairs.
   e. At all cantilever and moment connections.
   f. At all connections to plate girders and supporting connections.
2. Wind Connections: Seismic/Wind connections shall be fully tensioned with faying surface preparation class A. Bolts are designed as bearing connections. Wind connections shall be used at all beams, braces and columns in braced frames or moment frames and where noted on the plans.

   These connections do not need to be designed as Slip-Critical unless noted on the plans.

3. Pre-Tensioned Bolted Connections: Pre-Tensioned Bolted Connections shall be fully tensioned and used in the following locations:
   a. Wherever required in the AISC Provisions
   b. Wherever noted on the drawings
   c. For all bolts with tension loads (hangers or braces)
   d. Snug-Tight Bolted Connections: Snug-Tight Bolted connections may be used where permitted by AISC provisions, and at locations not noted above. All bolts shall be bearing bolts (N - threads included) unless specifically noted as slip-critical (SC).

4.e.5 Anchor Rods

ASTM F1554 Grade 55

fy = 380 MPa (55 ksi), with supplemental requirement S1 for weldability

fu = 517 MPa (75 ksi)

4.e.6 Post-Installed Anchors

Anchors shall be designed based on the provisions of ACI 318M-11 and also by the “HILTi methodology” or any corresponding propriety material procedure as described here below.

Anchors: “HILTI” HAS-E (ISO 898 Class 5.8) threaded rods or equivalent

Preferred sizes: 20mm Φ minimum for general use

Adhesive:	HILTI HVU capsules for general use or equivalent
	HILTI HIT HY 20 with screen tube for hollow masonry or equivalent
	HILTI HIT HY 150 for solid masonry or concrete or equivalent
	HILTI HIT RE 500 for solid masonry or concrete or equivalent

4.e.7 Stud Shear Connectors

Type: Headed shear studs conforming to AWS D1.1, Type B

fy = 345 MPa (50 ksi), fu = 450 MPa (65 ksi)

4.e.8 Welding Electrodes

Grade: E70xx (fu = 480 MPa [70 ksi])

Minimum weld size: 6mm (1/4") UON; see also AISC

4.e.9 Post-Tensioned Concrete

This section is applicable if bonded PT is chosen as an option. The following properties may be assumed in post tensioning design options:

Concrete:

Characteristic Strength	f'c = 40 to 70 N/mm2
Strength at Transfer	fcu,i = 25 N/mm2
Elastic Modulus of concrete	Ec = 0.043 x wc1.5 x sqrt (f'c)

All above values are to be confirmed by the post tensioning concrete contractor

Bonded Tendon Properties:

Prestressing strand	(ASTM A416M, Grade 1860 7-wire strand low relaxation)
Ultimate strength	fu = 1860 N/mm2
Wire superstrand, low relaxation	12.7mm, 12.9 mm and 15.2 and 15.7 mm diameter
Yield Strength	1680 N/mm2

Jacking Force fpi = 80% jacking force (1488 N/mm2)

Losses As calculated

Cover to ducts are expected to be as per the following table:

Location	Cover (mm) including installation tolerance
Internal concrete above Ground level (Tower Structure) and fire rating	Durability cover, fire cover, installation tolerance = total cover to duct
Continuous Slabs (2 h)	20, 19, 5 = 25
Simply Supported Slabs (2 h)	20, 19, 5 = 25
Continuous Beams (2 h)	40, 40, 5 = 45
Simply Supported Beams (2 h)	40, 40, 5 = 45
Transfer beam (4 h)	40, 48, 5 = 55
Concrete above Ground level (Parking Structure) if present	
Continuous Slabs (2 h)	25, 19, 5 = 30
Simply Supported Slabs (2 h)	25, 19, 5 = 30
Continuous Beams (2 h)	40, 40, 5 = 45
Simply Supported Beams (2 h)	40, 40, 5 = 45
Transfer beam (4 h)	40, 48, 5 = 55

4.e.10 Masonry

4.e.10.1Normal Weight

(Only where light weight AAC blocks cannot be provided.)

- Concrete block: ASTM C90 Grade N1
- Compressive strength = 19.3MPa, (2800 psi)
- Mortar: ASTM C270 Type M
- Grout: f'c = 3.8MPa, (2000 psi)
- Masonry strength: f'm = 13.8MPa, (2000 psi)

Strength determination: Unit strength method, or Prism test method

4.e.10.2Autoclaved Aerated Concrete

- Concrete block: ASTM C1693
- Density: 700 kg/m3
- Compressive strength = 2 to 7 MPa (300 to 1000 psi)

4.f Loading Data

General design criteria for dead and live loads are given in this section. Detailed loading plans including schedules and plan drawings shall also be provided in addition to the structural design criteria.

4.f.1 Gravity Loads

4.f.1.1 Dead Loads - General

Dead loads have been calculated using the following densities:

Material Self Weight	[metric]
Concrete (normal weight including allowance for reinforcement):	24.5 kN/m3
Concrete (lightweight including allowance for reinforcement):	18 kN/m3
Steel:	78.5 kN/m3
Norma lweight concrete block density:	20 kN/m3
AAC block work (for internal uses):	7 kN/m3
Soil for planters (saturated):	19 kN/m3
Compacted soil for traffic	22 kN/m3
Screed and hard floor finishes	20 kN/m3
Water	10 kN/m3
Float glass	25 kN/m3

The figures above are maximum weights that are used in the design of the building structure; the cladding consultant may select lighter systems, but should inform the engineer.

Cladding: (provided in terms of vertical surface area)

Material	Recommended values
GRP Composite Cladding Panels	To be calculated as per actual conditions
Double Glazed Facade	To be calculated as per actual conditions

Note: Indicative values of façade will be in the range of 2.5 kN/m.

4.f.1.2 Superimposed Dead loads – general

The following recommended values for superimposed dead loads (see section 5.4 for partition loads). The designer must confirm these load values with respective project requirements

Lobby / retail levels	Recommended values
Ceiling / Services	0.8 kPa
Finishes (as per project requirement)	as per project requirement
Wall Partition up to 4.0m maximum height (based on single 200mm skin aerated lightweight block with cavity for demise walls)	1.64kN/m per meter high
Wall Partition more than 4.0m high	As per blockwork design

Lobby / Amenity levels	Recommended values
Ceiling / Services	0.8 kPa
Finishes	as per project requirement
Wall Partition (based on single 200mm skin AAC block for demise walls)	1.64kN/m per meter high
Future Partition Contingency	1.0 kPa (on plan)

Parking levels	Recommended values
Services	0.5 kPa
Curbs	0.5 kPa

Residential (Freehold and serviced apartments)	Recommended values
Ceiling / Services	Generally 0.5 kPa. Evaluate if 0.35 kPa can be used)
Finishes (as per project requirement). Refer to recommendation in section 4.4.2 (50mm total build-up).	1.0kPa
Demising Wall Partition (based on single 200mm skin aerated lightweight block with cavity for demise walls) or alternative using light weight panels	1.64kN/m per meter high but to be checked as per project requirements for optimization
Internal Partitions: Two options available. Consult with EMAAR on the issue. 1. Lightweight AAC block wall (single 100mm lightweight aerated block for internal partitions – single skin 13mm plasterboard each side) 2. Internal partitions (Refer to section 4.3.1)	 0.94 kN/m per meter high Minimum 1.0kPa addition to floor Live Load

Club / Penthouse	Recommended values
Ceiling / Services	0.5 kPa
Floor finishes	As per project requirement
Wall Partition	See retail section
Future Partition Contingency	1.0 kPa (on plan)

Office areas	Recommended values
Ceiling/Services	0.5 kPa
Services (when ceiling is not provided as per Architectural requirement)	0.3 kPa
Finishes	As per architectural requirements.
Moveable Partition	Minimum 1.0kPa addition to floor Live Load

Mechanical Floors	Recommended values
Plinths/supports (100mm high concrete)	2.4 kPa
Ceiling/Services	2.0 kPa
Acoustic Slab (150mm)	3.6 kPa
Data Centre / IT / LV / UPS Rooms	**Recommended values**
Ceiling	0.3 kPa
Services	0.5 kPa
Finishes (minimum recommended)	3.0 kPa
Partitions	Minimum 2.0kPa addition to floor Live Load

Storage areas	Recommended values
Ceiling	0.3 kPa
Services	0.5 kPa
Finishes	as per project requirement

Toilets	Recommended values
Ceiling	0.3 kPa
Services	0.5 kPa
Finishes - Refer to recommendation in section 4.4.2	as per project requirement

Loading dock	Recommended values
Ceiling	0.3 kPa

Loading dock	Recommended values
Services	0.5 kPa
Finishes	as per project requirement

Core	Recommended values
Ceiling / Services	0.3 kPa
Finishes	as per project requirement
Partitions (to be considered only where concrete walls are substituted by partition walls, value to be multiplied by height)	Minimum 2.0 kPa

4.f.1.3 Live loads – general

Live loads are generally in accordance with ASCE/SEI 7-05

Live loads assumed for each occupancy are as follows:

Occupancy or use	Uniform load[1,6] kPa	Concentrated load[2] kN
Residential and Apartments Private rooms and corridors serving them Balconies (as per Dubai Municipality requirements) Public Rooms and corridors serving them	 2.0 3.0 4.79	
Assembly Areas and Theatres Fixed Seating Free assembly	 2.87 4.79	-
Lobbies	4.79 NR	-
Garages, Car Parking Passenger cars only Buses, Trucks, and mixed usage	 2.5[5] NR[9] Refer to IBC 2009	 13.35[2,5,9] Refer to IBC 2009
Areas for office use: Code minimum, no storage File rooms/Data Centre Copy rooms Corridors and Lobbies	 2.4 7.18 4.79 4.79 NR	 8.902 - 8.902 8.902

Occupancy or use	Uniform load[1,6] kPa	Concentrated load[2] kN
Swimming pools Swimming pool decks	As per calculated maximum water depth 4.79 NR	-
Roof Code minimum, flat Promenade	 1 2.87	 - -
Vehicular driveway, areas subjected to trucking	12.0 NR	Refer to IBC 2009
Stairs and Exits	4.79 NR	-
Inaccessible areas including Plenum spaces	1	
Public toilets	3.0	-
Storage Light Heavy	 6.00 NR 11.97 NR	 - -
Walkways and Elevated Platforms (other than exitways)	2.87	-
Dining Rooms, Restaurants, Club	4.79 NR	-
Retail	4.79	4.45
Mechanical Floors (based on usage)	4.79 to 12.0 NR	35.6
Sidewalks, vehicular driveways, and yards subject to trucking7	12	35.6
Skylobbies	4.79	4.45
Helipads8	Refer to IBC 2009, section 1605.4	
Tower Crane	TBC	

Notes:

1. Uniform loads marked "NR" are non-reducible.
2. Concentrated loads shall act over an area of 0.58 m2 unless otherwise noted. Wheel loads shall act on an area of 12,900 mm2. For stairs, concentrated loads shall act over 50mm by 50mm area. Concentrated loads need not be applied concurrently with uniform load.
3. The weight of landscaping materials is considered as dead load and is computed assuming saturated conditions. Landscaping loads to be based upon provided landscaping details.
4. Where clear height of garage entrance exceeds 2.13 m, loads for buses, trucks and mixed usage shall be used. Or actual wheel load increased 30 percent for impact, whichever is larger.
5. HS20-44 Truck loading, uniform load is applied over a 3 m width within a 3.66 m lane simultaneously with the concentrated load specified, 80 kN for moment controlled design and 115.7 kN for shear controlled design.
6. For flat slab and post-tensioned construction, a 2.0 kPa live load is to be used without any increase.

7. Any floor deemed to be accessible to emergency fire truck shall be designed using a minimum live load of 12kPa as per Dubai Municipality requirements. For design of columns carrying two or more stories, permanent loadings may be used.
8. Labelling of helicopter capacity shall be as required by the Dubai Municipality.
9. Members supporting two or more floors live loads may be reduced in accordance to ASCE 7

4.f.1.4 Live loads Reduction

Live loads may be reduced in accordance with IBC 2009.

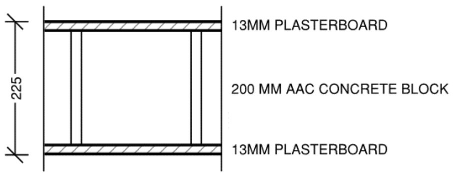
4.f.1.5 Partition Loads

The following partition loads are recommended in the residential floors. These are also included in the tables in section 6.1 Superimposed Dead Loads.

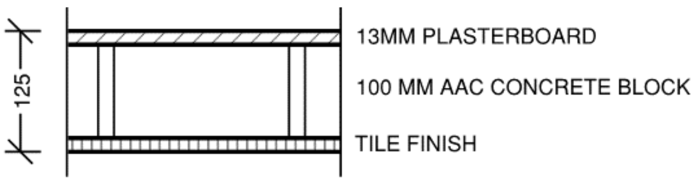
For demising walls: 1 layer of 200mm AAC blockwork and 13mm plasterboard on each face. Effective vertical weight of this wall type will be 1.64kPa on elevation. Alternatively, light weight panels can be used if they can meet the same requirements (acoustical etc).

For internal partitions except wet areas: 100mm thick wall with 13mm Gypsum board on each face attached to light metal stud framing with insulation. Effective vertical weight of this wall type will be 0.50kPa on elevation. Other lightweight panels systems can be proposed if they can meet the same requirements.

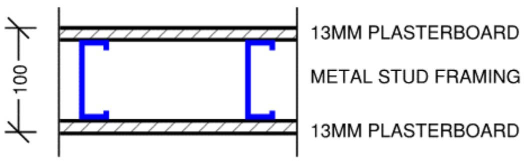
Internal partitions on wet areas (e.g. toilets, kitchen walls): 1 layer of 100mm AAC blockwork with tile finish on wet side and 13mm plasterboard on the dry side. Effective vertical weight of this wall type will be 1.06kPa on elevation. Alternatively, light weight panels can be used if they can meet the same requirements (acoustical etc).



Demising walls



Internal partition (Gypsum)



Internal partition (wet areas)

It is permissible to convert the total load of the partition walls to an equivalent pressure load. The total weight of all the wall load within a panel can be divided by the panel floor area but should not be less than 1 kPa. Partition loads acting on slab edges and beams may not be converted to an equivalent pressure load.

4.f.1.6 Vehicle Loads

Landscaped floors when found accessible to emergency fire truck shall be designed for load allowances for emergency fire vehicle access.

Refer to section 6.1.3 for fire truck loading. Vehicle access routes and the specific vehicles and loading criteria shall be confirmed

4.f.2 Wind Loads

Wind loads shall be determined in accordance with ASCE 7. Minimum wind loads on force-resisting system or components and cladding is 0.5 kPa.

If a wind tunnel test will be performed for these buildings, the parameters in accordance with relevant design codes may be used for preliminary design unless more detailed information is made available by the wind tunnel consultant in sufficient time.

4.f.2.1 Basic Parameters

Generally, the basic wind parameters given in the succeeding table will suffice for most building tall projects described in this Guideline. Refer to DM Circular 37.

Strength Wind Speed: Service Wind Speed:	38m/s (136 Km/h) 30 m/s (108 km/h)	Basic Wind Speed Based on 3 Second Gust at 10m for 50 year return period Service Wind Speed at 10m for 10 year return period
Building Risk Category:	I,II, III, IV	As per ASCE 7
Exposure Category:	C	As per ASCE 7
Wind Directionality Factor	Kd = 0.85	As per ASCE 7
Topographic Factor*	Kzt = 1.0	As per ASCE 7
Gust factor		As per ASCE 7 To be calculated accounting for preliminary dynamic properties of the building

* Specific project requirements may require different values based on the topography of the site.

4.f.2.2 Wind Loads based on wind tunnel test results

The wind tunnel results can be used in order to calculate directly calculate the member design forces, reactions, drifts and lateral acceleration. The corresponding return periods applicable to each check is given in the following:

For deflection/drift serviceability limit states, the wind tunnel test loadings will be calculated using a wind speed with a return period of 10 years (for drifts and deflections) as per ASCE-7 codes.

For other serviceability considerations, a return period of 50 years will be used.

For Ultimate Limit states, the wind tunnel test loading calculated using a wind speed with a return period applicable to the risk category of the structure.

For damping, it is recommended to use 1 - 2% for service and 1.5 - 2.5% for strength in concrete buildings, and 1% for service and 1.5% for strength in steel buildings.

4.f.3 Seismic Loads

Parameters to be used for seismic loading is given in the table below. This applies to inputs required for both static and dynamic forces.

Occupancy Category	As per use or occupancy	ASCE 7
Numerical Coefficient R (Ductility Factor)	R - project specific	ASCE 7
Seismic Amplification Factor	Ω_0 = 2.8 - Tower (Bearing Wall System)	ASCE 7
Site Soil Class	(to be confirmed following Site Investigation)	ASCE 7
PGA(g) for 5% critical damping and 760 m/s shear wave velocity	0.2 for site class (B)	ASCE 7
Seismic Importance Factor	Project specific	ASCE 7 table 1.5-2
Ss (g)	0.51 for Soil type S	ASCE 7
S1 (g)	0.18 for Soil type Sb	ASCE 7
Tl (s)	24 for Soil type B	ASCE 7
Seismic Displacement	In-elastic seismic displacement = 0.7 * R * Elastic Seismic Displacement	ASCE 7
Seismic Design Category	C	ACI 318M-11 Table R1.1.9.1

Notes:

1. The factors PGA, Ss and $S1$ shall be adjusted according to the nature of the soil by using $FPGA$, Fa , Fv (site amplification factors) and referring to tables (118-1, 114-1, 114-2) in the American code (ASCE7) consecutively

2. In case of estimating the shear wave velocity for rocks for site class (B) instead of measuring it, ($FPGA$ = Fa = Fv=1) shall be used according to (ASCE7). The adjusted factors PGA, Ss and $S1$ shall be labeled consecutively in ASCE7 as $PGAm$, SMs and $SM1$ according to the nature of the soil.
3. The factor $PGAm$ shall be used for the calculation of soil liquefaction analysis; the seismic force impacting the $PGAm$ factor in the emirate is (M 6.2).
4. Seismic mass to be used is 1.0 DL + 1.0 SDL + 0.25LL (storage) + 1.0 LL (mechanical / fixed)

4.f.4 Hydrostatic Loads and Earth Pressure

All foundation elements and retaining walls shall be designed by recommendations given in the geotechnical report. Hydrostatic uplift must be checked for structure envisaged to be completed when dewatering is designed to be turned off. The design water table shall be established in the Geotechnical Interpretive Report considering possibility of elevated water table during design life of the structure.

The following information shall be confirmed by the geotechnical report:

Active and passive pressures of the in-situ strata
Unit weight of in-situ material
Water table level to be used for design

For backfilled areas the following shall be considered and confirmed with the Geotechnical engineer:

Unit weight of backfill = 20.0kN/m3;
Coefficient of earth pressure at rest, K0 of compacted backfill = 0.50.

Any possibility of unbalanced soil lateral load must be considered in case of construction on an adjacent plot. Any retaining wall assumed to be subjected to lateral pressure during construction shall be designed according to possible surcharge and hydrostatic loading when found present. Bracing conditions of the retaining structure shall be carefully captured in the analysis and design.

Uplift due to heave shall also be considered for both permanent and temporary conditions whenever applicable.

4.f.5 Concrete Protection for Reinforcement

The table below lists the nominal cover to all reinforcement:

Location	Use Fire rating	Minimum Cover*, Mm (including 5 mm installation tolerance)
Concrete cast against and permanently exposed to earth	All (2 h)	75
Concrete exposed to earth or weather	All (2 h)	50
Concrete NOT exposed to weather or in contact with ground	Slabs (2 h)	25
	Beams (2 h)	40
	Walls (4 h)	25
	Columns (4 h)	40
	Transfer beam (4 h)	40

*Values based on ACI 216.1M-07 and ACI 318M-11. Fire rating requirements to be confirmed by Fire and Life Safety consultant and are provided in section 6.10 of this guideline.

4.f.6 Load Combinations

Load combinations shall meet the specifications of the IBC 2009 section 1605 or ASCE 7-05 Chapter 2. For seismic loading, reference shall be made to the requirements of UBC 97 for combinations involving seismic actions.

4.f.7 Performance Criteria

The performance criteria discussed here will be used during schematic design and may be enhanced per client or fabricator requirements. The limits discussed here combine code requirements and guidelines as well as Emaar's experience on past project performance.

4.f.7.1 General deflection limits

The following general deflection limits are given below based on IBC requirements:

Construction	Incremental deflection (*)	Long term deflection (SW+SDL+LL)
Roof Members:		
Supporting plaster ceiling	Span/360	Span/240
Supporting non-plaster ceiling	Span/240	Span/180
Floor Members:	Span/360	Span/240
Cantilevers	Span/180	Span/120

(*) Incremental deflection is the deflection happening after installation of the fragile members.

Stricter deflection criteria are required for the following conditions:

Beams and slab on the line of facade: for the Post Façade installation deflection (See 'X' of Stage 3 in sketches below) – Span/480 or 20mm (smallest) for normal spans and span/240 or 20 mm (smallest) for cantilevers which rules always have to be coordinated with the Façade consultant.

Note: There should be no maximum deflection considered for slabs or balconies more stringent than the values given in tables of clause 4.f.7.1 and 4.f.7.2.

4.f.7.2 Concrete Beam and Slab Deflections

Deflections shall be calculated based upon following approach. (based on ACI 318 Table9.5(b))

ACI Deflection Limits (Emaar modified)

Type of member	Deflection to be considered	Deflection Limitation
Flat roofs not supporting or attached to non-structural elements likely to be damaged by large deflections	Immediate deflection due to live load LL using cracked section*. Δ = Δ LLInst	Span/180 *note: ponding should be checked separately
Floors not supporting or attached to non-structural elements likely to be damaged by large deflections	Immediate deflection due to live load LL using cracked section*. Δ = Δ LLInst	Span/360
Roof or floor construction supporting or attached to non-structural elements likely to be damaged by large deflections (Brittle)***	The part of the total deflection occurring after attachment of nonstructural elements (sum of the long term deflection due to all sustained loads and the immediate deflection due to any additional live load) Δ = Δ100LT – Δ 3MoDL Where: Δ 100LT = Total long term deflection using cracked section*. See below. Δ 3MoCDL = Δ DL(1+λ) Total deflection including time dependant deflection at 3 months due to DL on lower bound cracked section**. λ defined below. DL = Dead load including superimposed dead load.	Span/480 (see above for beams supporting masonry/stone)

Type of member	Deflection to be considered	Deflection Limitation
Typical Slab Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections	Total Long Term Deflection based on 100% Live Load Cracked section. Δ LT = (λ+1)Δ DLInst + (λ+1) Δ SDL + (1-Ψ) Δ LLInst + Ψ (λ+1) Δ LLInst Where: Δ DLInstt = Instantaneous deflection due to self-weight using cracked section* based upon effect of self-weight only. Δ SDL = Component of Instantaneous deflection due to superimposed dead load using cracked section properties based upon effect of self-weight and superimposed dead load. Δ LL = Component of Instantaneous deflection due to live load using cracked section*properties based upon effect of self, superimposed and live loads. λ = Long term deflection multiplier per ACI 9.5.2.5 and ACI 435 Alternatively creep and shrinkage factor can be considered as per EC2 regulations Ψ = percentage of live load assumed to be sustained on the floor (minimum value should be 0.25)	Span/240

*Calculate cracked concrete stiffness for most loading conditions using total load.

** Calculate cracked concrete stiffness for long term deflection prior to attachment of non-structural items using a lower bound cracked concrete stiffness based on 1*DL load only.

***Brittle elements likely to be damaged by deflections include plaster ceilings, plaster partitions and large stone tiles. Gyp-board partitions and ceilings as well as normal sized stone tiles are generally considered flexible.

For the purposes of design, the floor finishes may be considered flexible.

Nowadays, most designers use propriety floor analysis and design software such as SAFE, ADAPT, RAM etc. These software packages are built with sophisticated analysis capabilities that consider material nonlinearity and long term effects. These programs require the following values to carry out analysis for long term deflection:

- Creep coefficient
- Shrinkage strain, εsh

When these software are to be used to determine deflections, reference can be made to ACI 209.2R and ACI 435R-95 – alternatively creep and shrinkage factor can be considered as per EC2 regulations - to determine the above mentioned parameters.

For SAFE, Modulus of rupture to be used shall be in accordance to ACI 318, using the formula 0.50 sqrt (fc'). This accounts for:

- the presence of restraint stresses
- allowance to use higher modulus of rupture for normal density concrete (ACI 363)

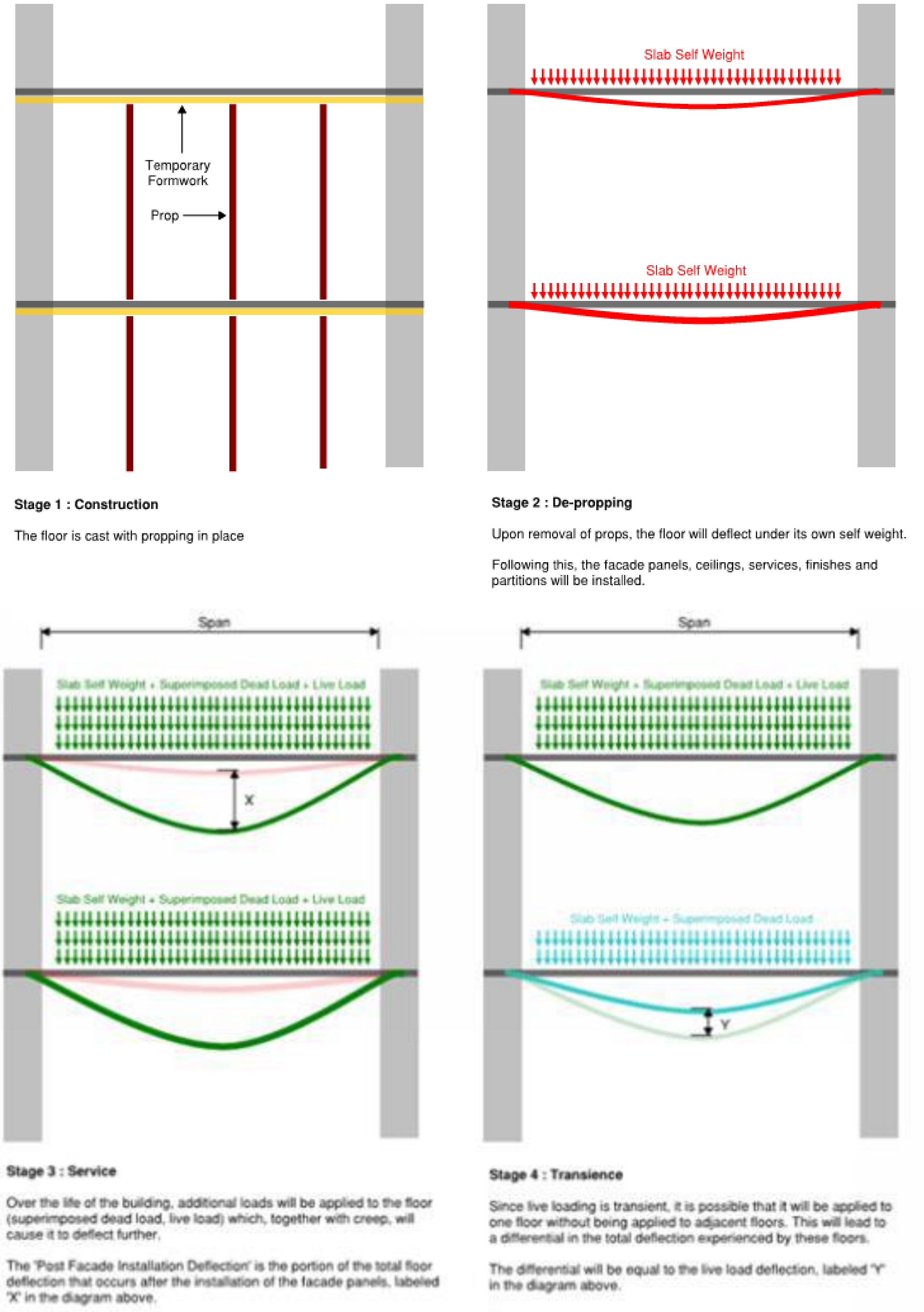
In these softwares, the anticipated tension and compression reinforcement shall also be considered while calculating deflection (further elaborated upon in separate document "Modeling guidelines").

Camber should be calculated based on DL deflection using DL effective section properties (cracked if DL causes section cracking). However, due consideration needs to be given to incremental deflection as in many cases, the latter rules the design when camber does not carry much benefit.

Floor deflections have to fulfil deflection limits on the line of the facade as detailed above focusing on the particular floor only. Beyond this check additional study to be carried out for the inter-storey differential floor deflection where façade is connecting adjacent floors focusing on cases where there is change in use of adjacent floors or significant load change is expected during the life time. Such limitations to be coordinated with the Façade consultant.

Deflection plots in many software show deflection even at the support face which is not realistic and to be deducted during the study.

For incremental deflection, relative deflection is to be considered: for instance for balconies cantilevering from a beam, the deflection of the beam at the balcony/beam joint is allowed to be deducted from the deflection measured at the tip of the cantilever.



4.f.7.3 Differential Settlements

For differential settlement between columns, settlements that do not induce floor gradients greater than that described by the above limitations will be considered acceptable. Differential settlement and span deflections shall be considered together. Façade consultant shall work with the structural engineers to confirm suitable façade movement allowance.

4.f.7.4 Building Sway

The total building sway shall be calculated from H/400 to H/600 for a 10 year return period wind loads (H= total building height) under the load combination DL + 0.5 LL + W10 yr.

Gravity drift will be calculated using construction sequence.

Note that this sway limit will need to be considered in conjunction with acceleration criteria and other limitations such as MEP services, Vertical Transportation tolerances and facade. Any such limitations will need to be provided by the relevant consultant for consideration in the structural design.

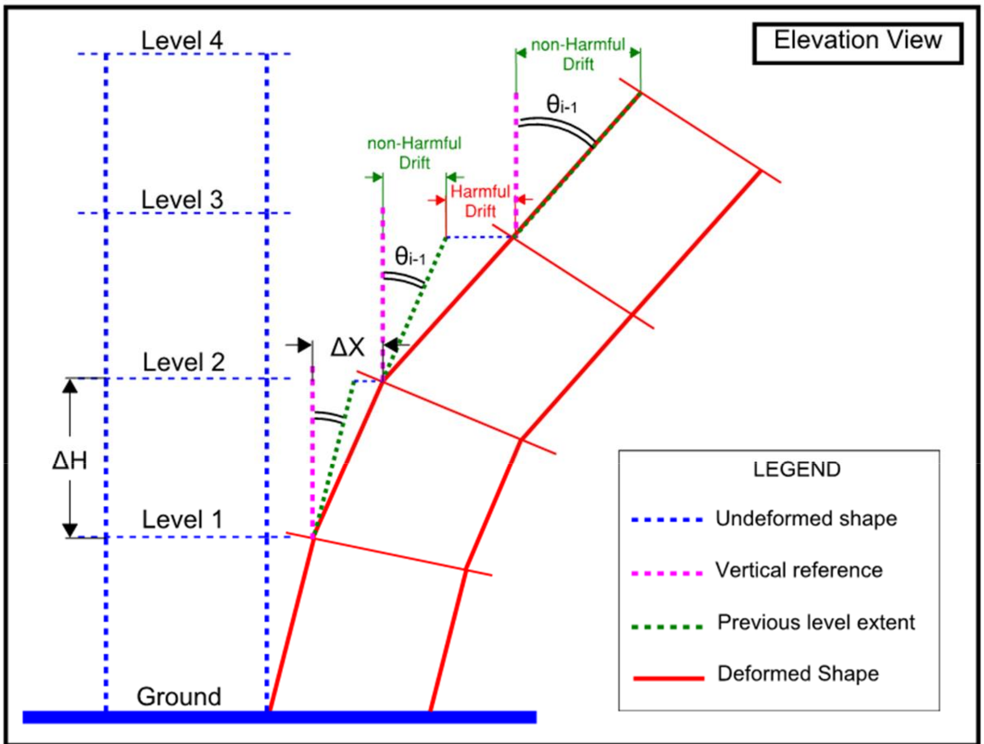
Where required, building sway will be limited to ensure no interaction with adjacent structures.

4.f.7.5 Interstorey Drift

For structures subject to wind loads, the inter-storey drift shall be calculated from h/400 to h/600 for a 10 year return period wind loads (h = storey height) under the load combination DL + 0.5 LL + W_{10} yr.

Harmful and harmless interstorey drift can be considered in which:

- Harmful (caused by flexural and shear deformations)
- Harmless (cuased by cumulative angular building curvature)



Using Secant Method as per Reference guidance

Total Inter-story drift = ΔX

Harmless Inter-story drift = $(\theta_{i-1}) * \Delta H$ ← in Radians

Harmful Inter-story drift = Total - Harmless = $\Delta X - \{ (\theta_{i-1}) * \Delta H \}$

For steel structures subject to wind loads, the inter-storey drift is limited to h/400 for sway frames.

For structures subject to seismic loads, the inter-storey inelastic drift is limited to h/50 (h = storey height).

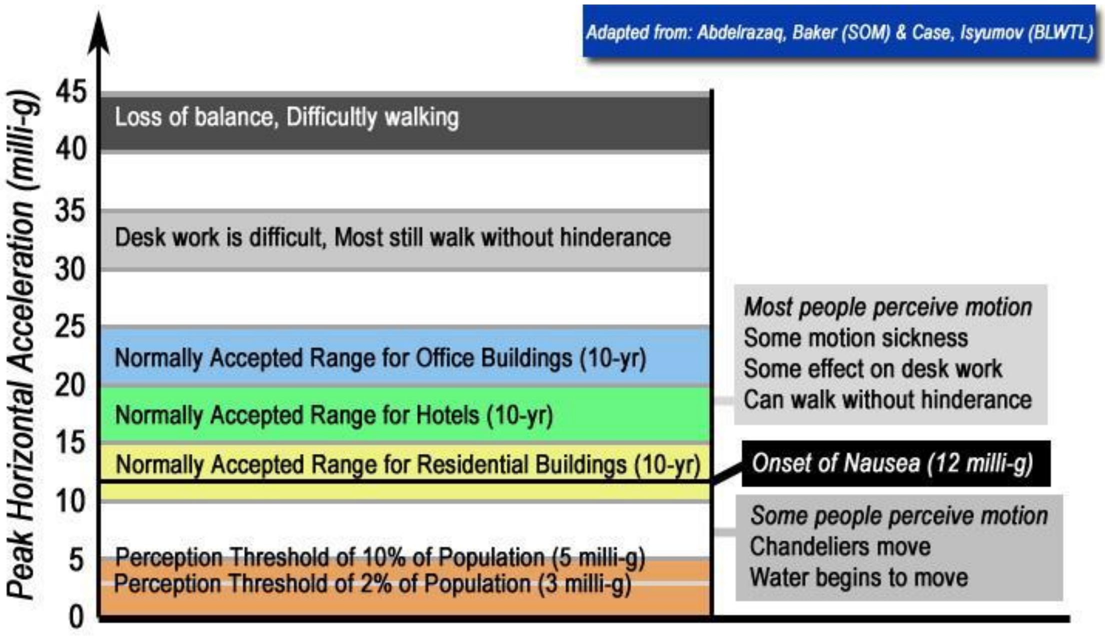
4.f.7.6 Building Separation Limits

Adjacent buildings on the same property shall be separated by at least ΔMT as per ASCE.

4.f.7.7 Lateral Acceleration

Wind induced lateral movements are important in the design of tall buildings, perception of motion is a complex phenomenon and affects people to different degrees as well as being dependent upon such factors as having a reference (horizon) and the environment (for example a busy office compared to a hotel room).

A common measure of probability of problems with perception of building movement by the building occupants is acceleration, with the onset of perception being as low as 3 to 5 milli-g (1 milli-g being 1/1000th the acceleration of gravity) for between 2 and 10% of the population. Torsional velocities are also considered following recommendations published by the Council on Tall Buildings and Urban Habitat (CTBUH). Refer to figure below for details.



Some published guidelines are available.

For acceleration limits of critical locations, ISO 10137 should be used as the basic criteria. The limit is based on a 1-year return period and is a sliding scale based on building period.

The 10 year return period acceleration limit criterium is 15 to 18 milli-g for residential occupancy. However, this is of secondary importance compared to the above.

Other areas with different usages may have more stringent requirements of acceleration limits and these will also need to be considered by the wind tunnel testing.

When wind tunnel testing is carried out, reference shall also be made to the recommendations and conclusions presented in the wind tunnel report for acceptability of anticipated values.

4.f.8 Floor Vibration

For steel beams, the natural frequency shall be minimum 3.0 Hz unless evaluated in accordance with SCI P354- Design of Floors for Vibration: A New Approach.

For PT floor not following the recommendation of Concrete Society TR43 2. edition Table I, a full dynamic assessment should be carried out following the method outlined on Concrete Society TR43 2.edition Appendix G.

6.5Hz for a reinforced concrete structure

7.0 Hz for a prestressed concrete structure
7.5 Hz for composite structures

8.0 Hz for steel structures

4.f.9 Durability

Concrete deterioration due to steel rebar corrosion is often mitigated by proper selection and installation of waterproofing systems. However, issues with basement leakage were observed in some projects even though waterproofing was installed. It is therefore recommended in this Guideline that in conjunction with waterproofing, minimum recommended requirements for concrete grades, concrete mixes, cover and crack widths are given in this section.

4.f.9.1 Crack width

Minimum crack widths are based on following provisions for super- and substructure elements.

Exposure condition Superstructure elements	Crack width mm
Not visible – internal environment	0.40 (1)
Visible and/or located in aggressive environment (car park, etc)	0.30 (2)
Water retaining elements (water tanks, pools, etc)	0.20 (3)

Exposure condition Substructure elements	Crack width mm
Not in contact with liquid	0.30 (2)
In contact with liquid (raft slab, pile caps, retaining walls)	0.20 (3)

Notes:

1. as per ACI 318-08. Deemed to be satisfied by ensuring minimum reinforcing requirements of the code are satisfied
2. as per recommendations in BS8110-2:1985 Clause 3.2.4
3. The serviceability requirements of EN 1992-3:2006 and Ciria guide C660 are adopted for a Tightness Class 1, which allows a small amount of leakage, surface staining or damp patches. Refer to Ciria C660 indicating Tightness Class and leakage requirements.

Limiting crack width is chosen based on Figure 2.2 from Ciria Guide C660 for a given pressure gradient (hp/h - Pressure head/Section thickness).

For tension piles, following needs to be met:

- Limiting the tensile stress to 140 MPa
- Limiting the crack width to 0.2mm considering the tension load

4.f.10 Fire Resistance

4.f.10.1 Fire Resistance Rating requirements for building elements

Fire resistance rating of structural elements for Type I (concrete) and Type II (steel) construction types shall comply with Table 1.6 of the UAE Fire and Life Safety Code of Practice. Where conflicts arise between fire resistance rating requirements of Table 1.6 of the UAE Fire and Life Safety Code of Practice and other fire resistance rating requirements in Chapter 1 of the UAE Fire and Life Safety Code of Practice, Table 1.6. shall be superseded. For example, exterior non-load bearing walls and interior non-load bearing corridor walls etc.

The Type of Construction required as outlined by Table 1.6 of the UAE Fire and Life Safety Code of Practice is based on DCD building categories as defined in the UAE Fire and Life Safety Code of Practice and the height and depth of the building as outlined in Table 1.7. and Table 1.8 of the UAE Fire and Life Safety Code of Practice and any additional modifications as allowed by Chapter 1 of the UAE Fire and Life Safety Code of Practice. Construction type for residential, hotel and parking structures based on Table 1.7. and Table 1.8 of the UAE Fire and Life Safety Code of Practice is summarized below. The UAE Fire and Life Safety Code of Practice should be referred to for requirements of other occupancy types.

Fire resistance requirements shall be as confirmed by fire and life safety consultant for the project.

Table 1.6.: Fire resistance rating for construction types (Type I to Type II) in Hours

STRUCTURAL ELEMENTS	TYPE I (442)	TYPE I (332)	TYPE II (222)	TYPE II (111)	TYPE II (000)
1. EXTERIOR BEARING WALLS (NOT LESS THAN TABLE 1.3)					
Supporting more than 1 floor or columns or other bearing walls	4	3	2	1	0
Supporting 1 floor only	4	3	2	1	0
Supporting roof only	4	3	1	1	0
2. INTERIOR BEARING WALLS					
Supporting more than 1 floor or columns or other bearing walls	4	3	2	1	0
Supporting 1 floor only	3	2	2	1	0
Supporting roof only	3	2	1	1	0
3. COLUMNS					
Supporting more than 1 floor or columns or other bearing walls	4	3	2	1	0
Supporting 1 floor only	3	2	2	1	0
Supporting roof only	3	2	1	1	0
4. BEAMS, GIRDERS, TRUSSES AND ARCHES					
Supporting more than 1 floor or columns or other bearing walls	4	3	2	1	0
Supporting 1 floor only	2	2	2	1	0
Supporting roof only	2	2	1	1	0
5. FLOOR-CEILING ASSEMBLIES	2	2	2	1	0
6. ROOF-CEILING ASSEMBLIES	2	1.5	1	1	0
7. INTERIOR NON-BEARING WALLS	0	0	0	0	0
8. EXTERIOR NON-BEARING WALLS (NOT LESS THAN TABLE 1.3)	0	0	0	0	0

Table 1.7.: Types of Constructions based on Civil Defence Building categories and occupancy types.						
OCCUPANCY	**LOW DEPTH UNDER-GROUND BUILDINGS < 7 m IN HEIGHT**	**HIGH DEPTH UNDER-GROUND BUILDINGS > 7 m IN HEIGHT**	**LOW RISE BUILDINGS < 15 m IN HEIGHT**	**MID RISE BUILDINGS 15 m— < 23 m IN HEIGHT**	**HIGHRISE BUILDINGS 23 m— < 90 m IN HEIGHT**	**SUPER HIGH RISE BUILDING > 90 m IN HEIGHT**
9. RESIDEN-TIAL GROUP A, C	TYPE I (442) S, AUL NS, AUL TYPE I (332) S, AUL NS, AUL TYPE II (222) S, AUL NS, AUL	TYPE I (442) S, AUL, FUL TYPE I (332) S, AUL , FUL TYPE II (222) S, AUL , FUL	TYPE I (442) S, AUL, FUL NS, AUL, FUL TYPE I (332) S, AUL, FUL NS, AUL, FUL TYPE II (222) S, AUL, F12 NS, AUL, F11 TYPE II (111) S, 6690 m^2, F5 NS, 2230 m^2, F4	TYPE I (442) S, AUL, FUL NS, AUL, FUL TYPE I (332) S, AUL, FUL NS, AUL,FUL TYPE II (222) S, AUL, F12 NS, AUL,F11 TYPE II (111) S, 6690 m^2, F5 NS, 2230 m^2, H20m	TYPE I (442) S, AUL, FUL TYPE I (332) S, AUL, FUL TYPE II (222) S, AUL, F12 TYPE II (111) S, 6690 m^2, H26m	TYPE I (442) S, AUL, FUL TYPE I (332) S, AUL, H128m
10. RESIDEN-TIAL GROUP B	TYPE I (442) S, AUL NS, AUL TYPE I (332) S, AUL NS, 2418 m^2 TYPE II (222) S, AUL NS, 2418 m^2	TYPE I (442) S, AUL, FUL TYPE I (332) S, AUL , FUL TYPE II (222) S, 3627 m^2, FUL	TYPE I (442) S, AUL, FUL NS, AUL, F3 TYPE I (332) S, AUL, FUL NS, AUL, F3 TYPE II (222) S, AUL, F12 NS, AUL, F3 TYPE II (111) S, 5441 m^2, F3 NS, 2230 m^2, F3	TYPE I (442) S, AUL, FUL TYPE I (332) S, AUL, FUL TYPE II (222) S, AUL, F12 TYPE II (111) S, 5441 m^2, H20m	TYPE I (442) S, AUL, FUL TYPE I (332) S, AUL, FUL TYPE II (222) S, 6690 m^2, F12, H55	TYPE I (442) S, AUL, FUL TYPE I (332) S, AUL, H128m
14. HOTEL	TYPE I (442) S, AUL NS, AUL TYPE I (332) S, AUL NS, AUL TYPE II (222) S, AUL NS, AUL	TYPE I (442) S, AUL, FUL TYPE I (332) S, AUL, FUL TYPE II (222) S, AUL , FUL	TYPE I (442) S, AUL, FUL NS, AUL, G TYPE I (332) S, AUL, FUL NS, AUL, G TYPE II (222) S, AUL, F12 NS, AUL, G TYPE II (111) S, 6690 m^2, F5 NS, 2230 m^2, G	TYPE I (442) S, AUL, FUL TYPE I (332) S, AUL, FUL TYPE II (222) S, AUL , F12 TYPE II (111) S, 6690 m^2, F5	TYPE I (442) S, AUL, FUL TYPE I (332) S, AUL, FUL TYPE II (222) S, AUL, F12, H55 TYPE II (111) S, 6690 m^2, H26m	TYPE I (442) S, AUL, FUL TYPE I (332) S, AUL, H128m
19. PARKING STRUCTURES	TYPE I (442) S, AUL NS, AUL TYPE I (332) S, AUL NS, 2418 m^2 TYPE II (222) S, AUL NS, 2418 m^2	TYPE I (442) S, AUL TYPE I (332) S, 6696 m^2 TYPE II (222) S, 5441 m^2	TYPE I (442) S, AUL NS, AUL TYPE I (332) S, AUL NS, 2418 m^2 TYPE II (222) S, 5441 m^2 NS, 2418 m^2	TYPE I (442) S, AUL NS, AUL TYPE I (332) S, 4464 m^2 NS, 2418 m^2 TYPE II (222) S, 5441 m^2 NS, 2418 m^2	TYPE I (442) S, AUL, FUL TYPE I (332) S, AUL, FUL TYPE II (222) S, AUL, F12, H55	TYPE I (442) S, AUL, FUL TYPE I (332) S, AUL, H128m

Note:

- Residential Group A – Residential building, residential apartments, assisted living home
- Residential Group B – Staff accommodation
- Residential Group C – Labor accommodation

4.g Geotechnical Engineering Requirements

4.g.1 Introduction

The overall aims of this section are to provide requirements for, and an understanding of, the geotechnical design process, aid identification and minimization of risks, and ultimately enable cost saving for the final substructure design.

It is a basic premise of this section that these aims can best be achieved by following a standardized sequence of actions, the findings or results of which are recorded in a series of reports. This section is therefore written in the form of a summary of the contents of these reports, and these contents will also identify the actions that need to be carried out to provide the material for the reports.

Details of the actions and reports may vary according to the nature of the project for which they are applied. For a small simple project, it may be possible to merge some of the reports: conversely, for a large complex project it may be necessary to have a number of separate reports dealing with separate aspects.

It is important to be aware that geotechnical risk can never be entirely eliminated. However, a planned programme of information gathering and review, particularly at an early stage in a project, can greatly reduce the likelihood of later surprises, which can often cause significant delays and additional costs.

4.g.2 Outline of Process

The sequence of processes and reports described in this report is as follows:

1. Appointment of a geotechnical consultant for the project.
2. Appraisal Report, including site specific investigation requirement, by geotechnical consultant.
3. Factual Report by ground investigation contractor.
4. Interpretative Report either by geotechnical consultant, or by ground investigation contractor where his report is reviewed by the consultant.
5. Design Report by geotechnical consultant.
6. Instrumentation and Monitoring Plan by geotechnical consultant, or by enabling works contractor/ main contractor.

It is important that geotechnical design be carried out by staff familiar with the process and requirements. For small simple projects this may be a general civil or structural engineer with some geotechnical experience, but for large or complex projects the work should be carried out by a specialized geotechnical consultant or by the geotechnical section of a multi-disciplinary company.

The Appraisal Report describes the preliminary assessment of the geotechnical aspects of the project. Activities carried out as part of this assessment should include the gathering of easily available information (the “desk study”) and the planning of intrusive site investigation (e.g. boreholes, in situ soil and rock testing and groundwater investigation). For large or complex projects, it may be appropriate to decide to carry out preliminary intrusive investigations on a small scale, with the findings to be used to plan later more extensive investigations.

The Factual Report describes the findings of the intrusive site investigations. It is usually written by the Contractor who has carried out the work. For large projects, in which different specialized investigation techniques may be employed, separate reports may be written by the separate specialist contractors.

The Interpretative Report, as the name suggests, interprets the findings described in the Factual Report. It should derive a ground model, based on the known geology and site investigation findings, and recommend design values for soil and rock properties. Important aspects of the interpretative report include a review of whether the findings of the intrusive investigation are consistent with the assessment made in the Appraisal Report, and a review of whether any potential problems have been identified that might need further investigation.

The Design Report uses the recommendations of the Interpretative Report to derive recommendations for the geotechnical design of the project. Almost inevitably, there may be some overlap between the contents of the Interpretative Report and the Design Report and it may be a matter of judgement as to which of these a particular item should be included in. For small projects, the Design report may be a component of the Interpretative Report.

Even after thorough site investigation and interpretation, there considerable uncertainty may remain concerning the expected behavior of the ground during or after construction. In such circumstances instrumentation and monitoring may be employed. The purpose of this may simply be to provide assurance that the ground behavior is within the limits expected (this is often done, for example, to reassure neighbors who are concerned about possible effects of the work on their own properties). Alternatively, in projects that are judged to involve particularly high risk, the results of monitoring may be used to control the construction process (the “observational method”).

4.g.3 Guidelines

4.g.3.1 Geotechnical Appraisal Report

The Geotechnical Appraisal Report sets out the justification for the scope of geotechnical surveys, detail processes for the management of geotechnical risks and opportunities, and the derivation of geotechnical parameters for the substructure design. The Geotechnical Appraisal Report should provide information on:

1. Site conditions which includes the geomorphology, hydrographical network and land use.

2. Ground conditions including geology, hydrogeology, hydrology, seismicity and contamination potential of the site.
3. Risk assessment where the ground hazards relative to the scheme is identified, assessed and the risk mitigation strategy prepared.
4. Scope of site investigation and surveys.

The Geotechnical Appraisal Report shall, as a minimum, include the following:

4.g.3.1.1 Site Conditions

Regional Geomorphology

- Topographical maps, aerial photographs.
- Maps of hydrographic networks, surface and embedded hydraulic works (e.g. streams, channels and trenches).

Historical and Current Land Use of the Project and Surrounding Sites

- Description of archaeological finds, if any.
- Historical land uses (maps, photographs) with reference to previous industrial uses, irrigation, petrol stations etc.
- Land-use data (maps, photographs etc) with reference to quarries, backfills etc.
- Records and maps of main utilities networks and underground structures.
- Records of existing gas stations, their locations and distances from the project site, years of operation, fuel tanks.
- Description of urban conditions (i.e. coverage and building density, infrastructures etc).

Site walkover Survey

- General site description and current site uses, including site access and site boundary, surface of site, building and hard standing materials, evidence of contamination, vegetation, asbestos, storage of materials and tanks.
- Services.
- Site geology – surface outcrops, exposures of soils/ rocks, excavations.
- Site topography – slopes, cuttings, embankments, mounds.
- Surface water bodies and courses.
- Hydrogeology – wells, abstraction, seepages, aquifers.
- Site surrounding – nature (i.e. industrial, residential, commercial etc.).
- Site photographs

4.g.3.1.2 Ground Conditions

Geology, Seismicity and Hydrogeology

- Description of the geology of the wider area (i.e. stratigraphy, tectonics).
- Geological, seismo tectonic, hydrogeological maps of the wider area.
- Records (borehole logs etc.) and maps of any available geotechnical surveys at project or surrounding sites.
- Description of the hydrogeological and hydrological conditions in the wider area.
- Records, maps, photographs of all shafts and water wells with locations, depths and their existing conditions, water level and pumps information.
- Seismicity of the wider area.

Anticipated Ground Conditions

- Previous ground investigations, list of work undertaken.
- Conceptual Ground Model, including groundwater level.
- Contamination potential.

4.g.3.1.3 Risk Assessment and Mitigation Strategy

Description of the scheme and construction methodology (e.g. open cut slope stability, shoring, dewatering, foundation etc.).

- Identification of geohazards relative to the scheme and proposed construction methodology.
- Potential consequences and impact due to the hazards.
- Likelihood of hazards occurring.
- Mitigation.

4.g.3.2 Site Investigation

This investigation shall provide sufficient data for the consultant to prepare the ground interpretative report and design. It will be carried out by a specialist contractor or contractors to a scope and specification prepared by the geotechnical consultant. The geotechnical consultant should also be involved in supervision of the site investigation field and laboratory work.

The ground properties to be measured, as a minimum, shall include drilling indices, characterization parameters, shear strength, compressibility (ground stiffness), dynamic shear modulus where applicable, groundwater level, piezometric pressures and initial earth pressure coefficient. Therefore, the site investigation should include the following as a minimum:

- Codes and Standards of Investigations.
- Scope and rational of the investigation
- Exploratory holes programme

- Ensure good core recovery. Where total core recoveries (TCRs) are low, adjustment of core lengths and drilling speed shall be altered to ensure high recoveries. In the event where TCR are below 50%, additional SPT tests or pressuremeter testing may be considered.
- In-situ tests (e.g. SPT, pressuremeter, permeability, geophysical testing etc.).
- Laboratory tests.
- Exploratory hole location plan.
- Monitoring.
- Other investigations and tests, where necessary. In cases where special requirements are needed (e.g. scaling factor for intact rock stiffness/ rock mass stiffness etc), it is the responsibility of the design team working with the geotechnical consultant to specify other tests as appropriate.

4.g.4 Geotechnical Factual Report

The Geotechnical Factual Report shall be prepared by the contractor who carried out the work and shall consist of items specified in the site investigation specification, and as a minimum, shall include the following:

- Summary of field work and laboratory testing undertaken.
- Description of equipment, tests, codes of practice used.
- Clear borehole location plan (with coordinates).
- Geological cross sections.
- Borehole, trial pit logs.
- Test results.
- Monitoring data.

4.g.5 Geotechnical Interpretative Report

The Geotechnical Interpretative Report may be written either by the geotechnical consultant or by the site investigation contractor to a scope compiled by the consultant, who also reviews the report. It should be noted that it is part of the requirement of the authorities that the interpretative report should be a section of an overall factual and interpretative report written by the site investigation contractor. However, at least one instance is known where it was considered acceptable for the consultant to write the interpretative report, which was subsequently adopted by the site investigation contractor and incorporated into the overall report.

The Interpretative Report sets out the design parameters to support the temporary and permanent substructure design. The report shall, as a minimum, include the following:

4.g.5.1 Introduction

- Outline of the design and construction of the scheme; including structural loading, foundation levels, open-cut, shoring, dewatering, etc.
- Sources of data used to provide ground model, making geotechnical design assumptions and assigning design parameters.

- A summary of ground appraisal findings, including scheme risk highlighted.
- A summary of former ground investigation(s) undertaken

4.g.5.2 Ground Conditions

- Summary of the investigation works (current) performed.
- Ground profile and description of ground conditions encountered, including where appropriate reasons for poor sample recovery (e.g. very loose deposits, karstification or voiding).
- Plan and cross sections of geological profiles.
- Interpretation of ground conditions as related to the design and construction of the works.

4.g.5.3 Ground Properties

- Presentation of drilling indices such as total core recovery, solid core recovery and rock designation quality versus elevation profiles.
- Presentation of ground properties measured in-situ and/ or in laboratory. The ground properties should correspond to succession of strata and shall, as a minimum, include the following:
- Characterization parameters such as specific weight, density, particle sizes, Atterberg limits, moisture content etc.
- Shear strength parameters.
- Compressibility (ground stiffness) parameters when loading and unloading.
- Dynamic shear modulus, where applicable.
- Groundwater level and hydrogeological conditions, peizometric pressures that shall be used in the temporary shoring and long term conditions, permeability.
- Earth pressure coefficients (for soil, rock or backfill).

4.g.5.4 Geotechnical Design Parameters

The design parameters shall specifically correspond to the construction and design methodology of the scheme, and shall, as a minimum, include the following:

- Procedure of determination of design parameters (e.g. procedure described in paragraphs 2.4.5.2 and 2.4.6.2 of Eurocode 7, using the characteristic values of geotechnical parameters).
- Well documented empirical correlations from sources based on local conditions etc.
- Design shear strength parameters of rock based on rock mass strength parameters (e.g. rock mass rating, geological strength index, unconfined compression strength etc.) from conventional theory must be presented.
- Correlations between intact rock samples measurement (e.g. ground stiffness measurements) and design parameters of rock mass shall be considered and justified.
- If anchoring is to be used, values of ultimate bond strength and working bond strength of anchor and surrounding ground shall be given.
- Determination of characteristic values of Sulphate, pH, classification of concrete, structural performance and resistance of chemical attack.

4.g.5.5 Preliminary Engineering Considerations

- Review of the findings in the light of expectations from the Appraisal Report, identification of any differences and assessment of the consequences;
- Outline of foundation options and preliminary foundation solution, including ground improvement where necessary.
- Preliminary allowable bearing pressure which considers the limiting settlement criteria for associated foundation suitable for the scheme.
- We note subgrade modulus reaction is not a generic value and to be provided in geotechnical design report (discussed in the following section).
- Indicative pile capacities and pile settlement of single isolated pile.
- We note that pile group settlement assessment shall be undertaken and included in geotechnical design report (discussed in the following section).
- Preliminary slope stability assessment shall be considered, where necessary.
- Preliminary ground anchor design, where necessary.
- Excavability of the ground shall be considered, where necessary.
- Seismic conditions, seismic site category and liquefaction potential.
- Groundwater control.
- Requirements for ground improvement where necessary
- Requirements for any further investigation deemed necessary.
- Outline recommendations for instrumentation and monitoring.

4.g.6 Geotechnical Design Report and Considerations

The Design Report shall be written by the geotechnical consultant. It should document the key design criteria upon which the detailed design has been based on and the design. It shall also provide details of software to be used in the design process. The following shall be considered and included in the report as a minimum:

4.g.6.1 Open-Cut Excavation

- Stability assessment of open-cut slopes.
- Assessment of seepage.
- Software utilized.

4.g.6.2 Shoring Wall, Propping, Dewatering

- The shoring wall and dewatering designs shall be considered as a whole, and not separate designs. The main reasons for this are:
  - The toe depth and structural section of the wall depend, inter alia, on the water pressures acting upon it;
  - The toe depth of the wall also depends in part on the requirement to exclude water from the site during excavation.

- A summary of geotechnical design parameters specifically for the shoring wall and dewatering designs shall be included.
- Excavation base stability calculation shall be included.
- The shoring wall design calculation with consideration of propping and de-stressing shall be included.
- Dewatering design calculation shall be included.
- Software utilized

4.g.6.3 Shallow Foundation and Raft Foundation

- A summary of geotechnical design parameters specifically for shallow and raft foundation designs and structural loading shall be included.
- Safe bearing capacity and allowable bearing pressure.
- Estimated settlement and method utilized for estimation.
- For raft foundation, provide subgrade modulus reaction taking into account the structural loading pattern, raft size/ shape and flexibility and building stiffness. We note the subgrade modulus reaction of a raft is generally not a single constant value. Depending on the complexity of the project and ground conditions, appropriate subgrade modulus values may be established from modelling of both the structure and ground, or iteration between structural and geotechnical models.

4.g.6.4 Pile Foundation

- A summary of geotechnical design parameters specifically for the pile foundation design, and structural loading shall be included.
- Basis of pile capacity (compression and tension failure mode) assessment, including method, theory of assessment and factor of safety adopted.
- Pile group efficiency.
- Basis of foundation stiffness and settlement (pile group) assessment, including method and software utilised. The stiffness of large structural foundations depends on both the stiffness of the ground and the geometry and stiffness of the foundation (and to an extent the building stiffness). Therefore, it may be necessary to iterate between the structural and geotechnical models.
- Design calculation. Below piles design requirements should be followed to ensure compliance with the design requirement for piles:
- For tension piles, crack width due to tension needs to be limited to:
    - 0.1 mm due to upward ground water pressure
    - 0.2mm considering secondary forces (wind, seismic, etc)
- Geotechnical design parameters to match with soil report recommendations
- Minimum percentage of reinforcement to be provided over full pile length to provide ductility
- Ensure that pile spacing is no less than 2.5 times the diameter of the piles
- Stresses: to be less than 25% of specified cube strength as per BS8004: 7.4.4.3
- General:
    - Ensure pile design allows for both gravity and lateral loads

- Minimum design of horizontal / lateral force: 5% of pile capacity
- Ensure piles are designed out of verticality (1/75)
- Out of position (7.5 cm)
- Ensure piles have a minimum FoS of 2.5 unless additional investigation studies and fiels tests are conduced
- Minimum socket of 3 times diameter in rocks
- Bentonite reduction factor

The above design requirements – H=5%V and out of positional moments - can be removed if following items are taken care of:

- Isolated temperature changes within raft, and temperature distribution from column to raft to be considered
- Detailed pile group assessment considering soil-structure interaction, building stiffness and foundation stiffness to be undertaken.
- Moments due to slab dishing should be considered.
- Kinematic effects of earthquake loading should be considered.
- Undertake sensitivity checks should piles be constructed out of position.
- Embedment of raft to be considered

However, for code compliance of structural design of piles, out of positional moments (by treating piles as single braced columns) should be considered as minimum moments and not additional moments. This means if moments due to horizontal forces M_H are greater than out of positional moments M_e, then M_H should be considered in design and vice versa. The current requirement is design moments = $M_H + M_e$.

- Preliminary pile test data, where available shall be interpreted and presented. Comparison between predicted and back analyzed pile shaft friction and end bearing resistance shall be presented along with settlement and elastic shortening of pile.

4.g.6.5 Permanent Basement Wall

- A summary of geotechnical design parameters specifically for the permanent basement wall design shall be included.
- Basis of design.

4.g.6.6 Sensitivity Analysis

- A sensitivity analysis of key design parameters shall be undertaken and included within the report.

4.g.7 Instrumentation and Monitoring Plan

The objective of an Instrumentation and Monitoring Plan is to set out guidelines to ensure that the project proceeds safely during the construction period. It provides further verification of the assumptions used in design and allows for early identification of any unforeseen behavior of the ground or neighboring structures.

In order to formulate a building damage control strategy due to the propose design and construction of the scheme, the following shall be included, where required:

- A condition survey of adjacent structure and utilities.
- Survey points for building movement (vertical and lateral) of adjacent structures.
- Survey points for settlement measurements of the new structure;
- Earth pressure gauges to measure pressures under foundations;
- Strain gauges to measure stresses in structural elements, e.g. piles.
- Monitoring of shoring wall and ground profile behind wall (e.g. with use of inclinometers, survey monuments etc.).
- Piezometers to measure groundwater levels within and outside the excavation.
- Building damage control strategy, including mitigation and remedial actions shall be prepared based upon observation of degree of ground and wall movements (i.e. trigger values).
- Trigger values where response and remedial actions may need to be actioned

4.h Post-Tensioned Design Criteria

Following design criteria are to be followed when designing PT slabs.

4.h.1 Calculations

- For slab design, if the slab is allowed to have torsion moments ("Twisting moments") in plan then this moment must be considered in the reinforcement design. Otherwise, the slab has to be considered as non-torsion slabs.
- Slabs which have a prestress level of over 2 MPa (as an average for a bigger area, not localized peak) or if floor dimension in one direction is more than 50 m or there is more than one stiff restraint, the following need to be checked for:
  - Shrinkage from early thermal effects
  - Creep (including shortening due to prestress force)
  - Drying shrinkage of concrete
  - Tendons with high frictional and wobble losses due to their profile and length shall be stressed from both ends
- If the average pre-compression exceeds 3.0 MPa, the design engineer shall explicitly recognize and account for the consequence of shortening of the member in connection with the restraint of the member's supports.

4.h.2 Serviceability Requirements of Flexural Members

Design for serviceability requirements of members shall be in compliance with the following:

- For post-tension elements: ACI318 standard

- For pre-cast prestressed elements: PCI

4.h.3 Permissible Stresses in Pre-Stressing Steel

The permissible tensile stresses in all types of pre-stressing steel, in terms of the specified minimum tensile strength f_{pu}, are to be summarised as follows:

- Jacking force for post tension elements: Shall not be more than 0.80 fpu
- Post-tensioning tendons, at anchorages and couplers immediately after force transfer: Shall not be more than 0.70 fpu

4.h.4 Flexural and Shear Strength

Flexural and shear strength of pre-stressed members shall comply with the following:

- For post-tension elements ACI318 standard
- For pre-cast prestressed elements: PCI

4.h.5 Minimum Bonded Reinforcement

A minimum area of bonded un-tensioned reinforcement shall be provided in all flexural members, as follows:

Negative moment areas at column supports in flat slabs	The minimum top non-prestressed bonded reinforcement A_s in each direction shall be computed by: $As = 0.00075\ A_{cf}$ Where A_{cf} is the gross cross-sectional area of the slab-beam strips in each of the two orthogonal equivalent frames intersecting at a column in a two-way slab This reinforcement shall be distributed between lines that are 1.5h, where h is slab thickness, outside opposite faces of the column support: "effective width" At least four bars shall be provided in each direction Spacing of bonded reinforcement shall not exceed 350 mm Minimum length of bonded reinforcement in negative moment areas shall extend one-fifth the clear span, on each side of the support
Bottom reinforcement	Minimum thermal and shrinkage reinforcement is 0.0018 (420/fy)Ac, but not less than 0.0014Ac for the floor (where fy is the rebar yield strength). It is not specified if it has to be top or bottom reinforcement, likely bottom for the mid span for shallow sections, while for deeper slabs definitely split between top and bottom Bottom mesh to run through column location. Bottom steel at columns and support locations should be not less than 30% of the required ultimate top steel at the same location.
Integrity reinforcement	At the supports, at least one tendon – the minimum being two strands – must pass through the columns or walls. The following shall be observed:

	If the tendon is not passing through the columns or walls, then a minimum amount of bottom reinforcement should be provided for structural integrity Bottom reinforcement should be no less than 1.5 times the minimum flexural reinforcement nor (2.1 bwd / fy) where bw is the width of the column face through which the reinforcement passes. Minimum extension of these bars beyond the column or shear cap face shall be equal to or greater than the bar development length. (Refer to ACI-318, latest version)

4.h.6 Deflection Control

4.h.6.1 Cracking Effects

Effects of cracking should be taken into account by modifying the stiffness - "EI" properties of the concrete for areas which have exceeded the allowable tensile stresses "if exceeding the tensile limits is allowed by the chosen design code". The modification factor to the EI value will vary depending on the extent of the cracking.

Most of the structures are a combination of RC, un-cracked PT and cracked PT sections such as un-cracked PT flat slab connecting to core wall with RC section or RC beam supporting the un-cracked PT slab. The design method is a must to capture the real behavior taking into consideration the combination of cracked and un-cracked sections.

4.h.6.2 Span to thickness ratio limit

The span-thickness ratio for pre-stressed slabs for floors generally should not exceed 40. In any case, structural calculations should verify that both short and long-term deflection, camber, vibration, frequency and amplitude are within the permissible limits which will allow to designer to reach even higher ratios.

Minimum thickness of main PT Slab to be 200mm. For balconies minimum 150mm thick PT slab can be used taking into consideration that such section depth can accommodate only PT dead end, but no live ends.

The PT system has to be able to fit within the particular slab depth considering the bursting reinforcement also with appropriate concrete cover.

4.h.7 Slab System

4.h.7.1 Pre-compression

For slabs with varying cross sections along the slab span, the tendons must provide an effective pressure of 0.9 MPa in one direction and 0.7MPa in the other direction as per code.

Shrinkage and temperature reinforcement (0.0018Ac x 420MPa/fy, but minimum 0.0014Ac) to be provided, where fy is the yield strength of the reinforcement and Ac is the concrete section area. For shallow slabs this reinforcement can be placed in the bottom layer, while for thicker section to be split between top and bottom layer. Minimum mesh bar diameter to be T10 and maximum spacing of the mesh to be 350mm. As minimum reinforcement fulfils the shrinkage and temperature requirements of a fully RC section there is no need any more to check the pre-compression in the design model from minimum pre-compression point of view.

As a first step of the design Nominal amount of PT to be placed in both directions. In one direction based on the 0.9MPa and in the other direction based on 0.7MPa as per the below formula:
"spacing of tendons in mm" = 1000mm / (("slab depth in mm" x "1000mm") x (0.9MPa or 0.7MPa) / (1116MPa x "area of strand in mm2 " x "strands per tendon")) where the nominal stress in a strand is 1116MPa if the strand jacking stress is 1488MPa.

As a second step provide sufficient PT to fulfil SLS and ULS design requirements in all design sections.

It might happen that a slab is PT in one direction and RC in the other direction. The best example is a one-way slab sitting on parallel walls, where PT tendons are needed perpendicular to the walls and PT tendons do not provide any benefit parallel with the walls, accordingly RC section to be designed where PT does not provide benefit and the RC section has to fulfil all RC requirements.

If the average pre-compression exceeds 3.0 MPa, the design engineer shall explicitly recognize and account for the consequence of shortening of the member in connection with the restraint of the member’s supports.

4.h.7.2 Ducts

Ducts for grouted multiple wire, multiple strand, or multiple bar tendons shall have an inside cross-sectional area of at least two times the cross sectional area of the pre-stressing steel.

4.h.7.3 Reinforcement detailing

For situations where it is not practically possible to place the pre-stressing tendons within 0.5h from the column, reinforcement should be placed to bridge the vertical force from the adjacent tendon to the columns.

Additional reinforcement is required where tendons are not within 0.5h from the column. The reinforcement should:

1. Be placed under the pre-stressing tendon
2. Have sufficient area to transmit the vertical component of pre-stressing for that tendon to the column
3. Extend a full anchorage length beyond the tendon

4. Lie within 0.5h of the column and at least one bar should pass over the column
5. Additional reinforcement is required in areas of congested electrical conduits, equivalent to the lost concrete cross sectional area (ideally away from column location)

Further, local bursting reinforcement is to be provided for all mono-strand tendon systems mainly in the form of helical bars. For multistrand systems, local bursting reinforcement is to be provided as per PT subcontractor specifications. Global bursting reinforcement is to be designed and provided due to the high concentration of the force and potentially high tensile stresses around the multistrand anchorage and between the multiple multistrand anchorages.

4.h.7.4 Edge Reinforcement

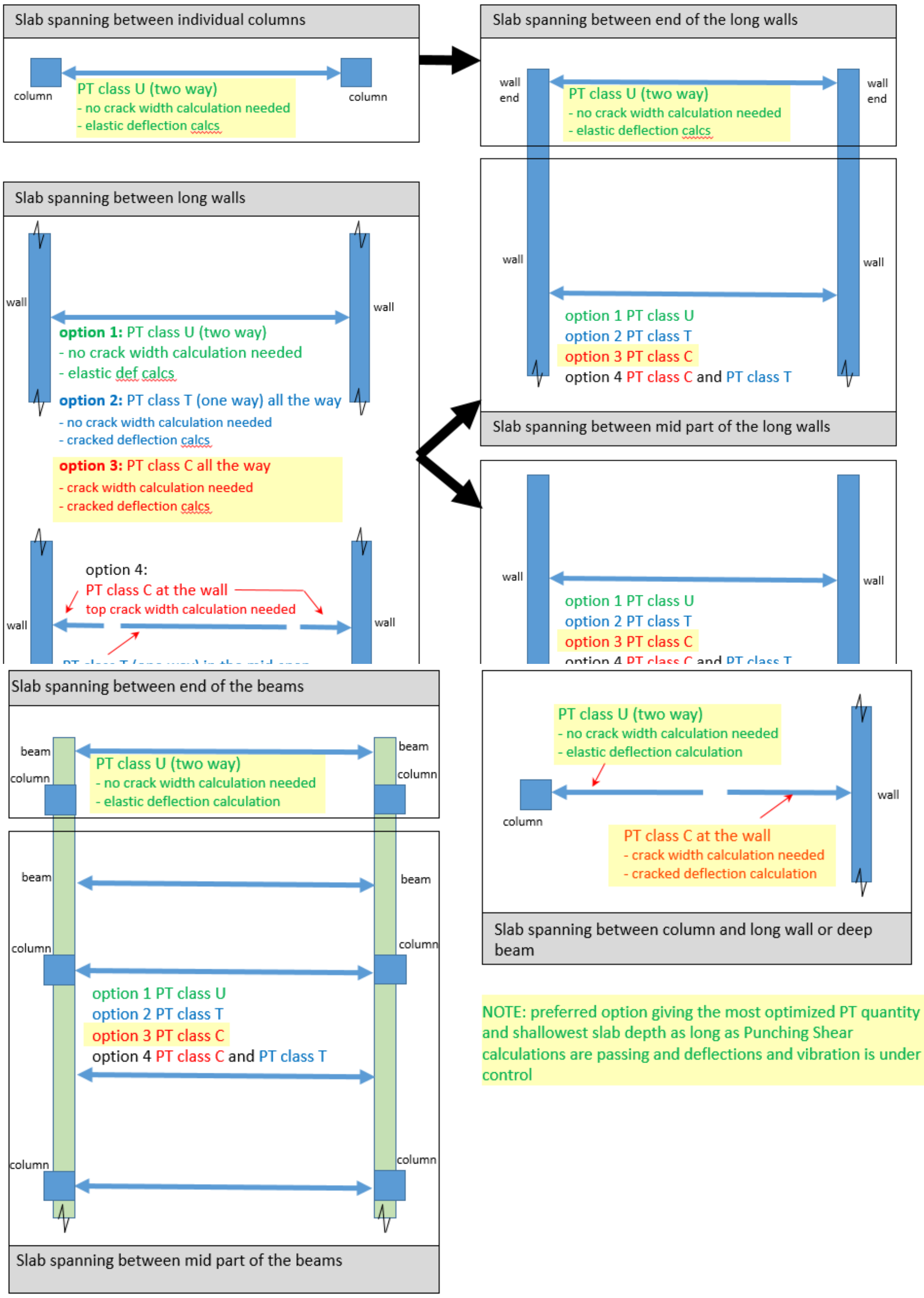
As the minimum reinforcement already covers the requirements of an RC slab (0.0018Ac x 420MPa/fy, but minimum 0.0014Ac) there is no need to detail any more reinforcement parallel with the slab edge or perpendicular reinforcement between the anchorages.

4.h.7.5 Punching shear

a. Post tensioned slabs are shallow and sensitive to punching shear, therefore detailed punching shear analysis to be provided for each individual column location with the final opening layout. Post tensioning pan boxes are to be kept away from punching shear zone or accounted for during the analysis as a week spot and required reinforcement designed accordingly.
b. Openings which are within 6D (where D is the slab depth) from column face to be taken into consideration in the PS analysis
c. A column to be considered as edge or respectively corner column if column face is less than 6D from the slab edge
d. Preferred to do the PS analysis with the FEM software used for the floor analysis which has all the relevant information available such as pre-compression, openings, slab edge distance, coexisting bending moment and reaction forces. Together these allow precise and economical design. Preferred software is RAM concept or SAFE.
e. Place at every column minimum 3 layers of PS reinforcement even if PS reinforcement is not required by gravity design.

4.h.7.6 Design strips / Integration strips

a. Below are examples of design strip settings for various cases of PT slab design



Conclusion: use appropriate class settings at all locations and run always load history deflection calculation which considers long term effects and considers level of cracking of each section automatically.

b. For RC section column and middle strip to be defined as per below detail

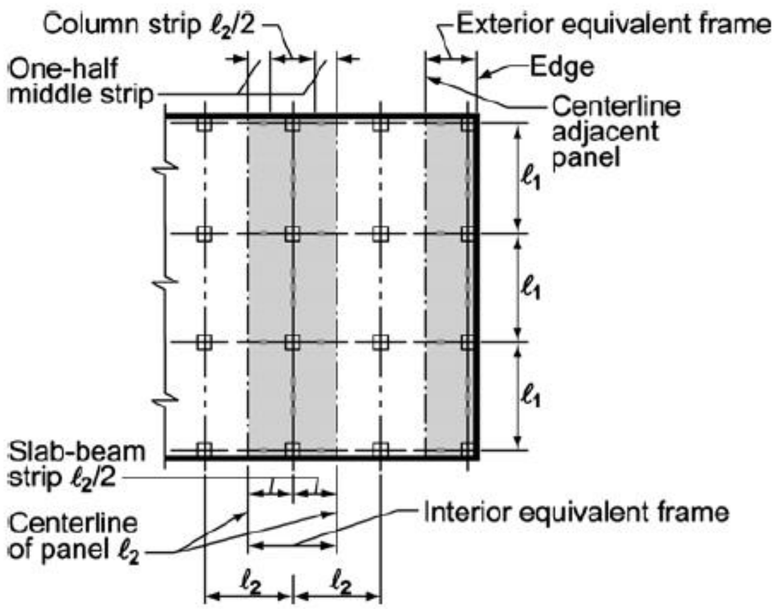


Fig. R13.7.2—Definitions of equivalent frame.

4.h.7.7 Pattern Loading

Pattern loading to be done for floor analysis only for cases defined in the ACI-318.

4.h.7.8 Conduits and Kink / Curvature of Tendons

Conduits and group of conduits are not allowed to be placed within the dead end zone and right behind the live end. Extent of the zone to be coordinated with the specialist subcontractor.

Curved tendons are generally to be avoided, but in case of difficulty to furnish the straight tendons, hair pins should be used in additional to bottom and/or top steel mesh and the deviation shall not exceed 1:12.

Curved tendons to be installed with progressive kink. If single live end is placed then kink is allowed in single direction, if double live end is placed then 2 directional kink is acceptable which is up to the judgement of the designer. Jacking sequence of the strands within the kinked tendon has to reflect the intent to avoid locking in the strands which can be achieved by stressing the shortest (most inside) strand, then working our way towards the most outside one within the same tendon. By default, in curved tendons, single stage stressing is a must and to do that, the concrete has to have sufficient strength so that the full jacking load can be carried immediately. The slab mesh has to fulfil the minimum shrinkage requirements of an RC section

4.h.7.9 Installation Tolerance of Tendons

a. Tendon vertical installation tolerance for all components (such as for live end, duct and dead end) is as follows:

- ±h/40 for slabs not more than 200mm thick

- ±5mm slabs more than 200mm thick
- for beams ±10mm and beyond 600mm depth use ±12mm

b. Horizontal installation tolerance is 100mm

4.h.7.10 Minimum Length of Tendons

Tendon length to be minimum 5m when wedge draw in is applicable

4.h.7.11 Tendon Spacing

Maximum tendon spacing 1.5m in both direction.

In the one direction c/c distance of tendons can be increased to 8D where D is the slab depth and not more than 1.8m if tendons cannot run closer due to obstructions such as openings. The minimum horizontal spacing between the ducts is the greater of 75 mm or duct width.

4.h.7.12 As-Built drawings and models of post tensioned structure

The updated model and drawings have to reflect all openings, broken strands, insufficient elongations, reduced concrete strength and all information which would be required to evaluate in the future if any change such as introduction of increased loads or introduction of new openings is acceptable.

4.i Third Party Requirements

It is the responsibility of Emaar to enter into an agreement for a detailed structural review by a third party consultant or peer reviewer for specific works, as listed in Section 4.i.1 of this Chapter. The main project consultant should advise Emaar in advance regarding the need for a peer review report. The peer reviewer shall share equal design liability with the main consultant.

4.i.1 Conditions and Requirements

Peer review is required for any of the following structures:

a. Specialized developments with new structural concept, features and design.
b. Irregular, complicated, huge, long and unusual structures.
c. Any building of a height more than G+40 floors. This requirement may be waived at the discretion of Authorities, depending on the engineer's qualifications and the prior experience and range of projects previously executed by the consultant.
d. For shoring work of four basements and above.
e. For shoring work of three basements and more located near to bodies of water.

Whenever there is any doubt about the need for a peer review for a project, it is the responsibility of the main consultant to raise a specific query by means of an official letter addressed to Authorities who will evaluate the need for a third party structural review on a case-by-case basis at the time of official submittal.

Authorities may request a third party review for any project if the main consultant and/or specialized subcontractor is found to be incompetent.

4.i.2 Minimum Qualifications and Eligibility Criteria

Authorities may reserve the right to approve or reject any proposed peer review consultant – applicable to both local and foreign consultants - based on their Dubai Municipality license, engineer's qualification(s), experience and past track record. The minimum qualifications and requirements of third party consultants or peer reviewers are as follows:

a. The third party consultant or peer reviewer shall have a minimum of ten years of experience in designing and reviewing similar projects, with a good track record. Authorities can reserve the right to request for credentials to be submitted.
b. Professional indemnity relating to the structural peer review and/or audit shall be as per the Dubai Economic Department (DED) and Dubai Municipality (DM) norms and requirements, and subject to any applicable laws, rules and regulations.
c. A minimum of three qualified, well-experienced, Dubai Municipality (DM) licensed structural engineers, with full knowledge of the international standards and codes for structural design and construction, as well as having prior experience of projects similar to that under review, must be available with the peer reviewer under their own sponsorship.
d. Peer reviewers shall use the internationally approved and regionally recognized industry standard software for all the structural analysis and design.
e. Spot checks by manual calculation shall be submitted for critical structural members and to validate the output of the software employed.
f. Peer reviewers are required to adhere to Authority's rules, guidelines, processes, procedures and latest circulars at all times and under all circumstances.

4.i.3 Scope of Work

The scope of work of the third party consultant or peer reviewer shall include, but is not limited to, the following:

a. To provide a detailed structural report including all major parameters and aspects of the structural review process.
b. To develop an independent structural model in a different software to that used by the main consultant.
c. To provide a report containing a full history of all key communication and correspondence between themselves and the main structural design consultant.
d. To honestly report any review findings that reveal deficiency or discrepancy in the structure.
e. To be in attendance at all meetings and technical discussions held between Authorities and the main structural design consultant.
f. To share equal design liability along with the main structural design consultant.

g. All design revisions made by structural members should be subject to review and approval by the third party reviewer.
h. All structural drawings shall be stamped by the third party consultant or peer reviewer.
i. Please note that any drawings without stamps will be rejected by Authorities.
j. The cover sheet of the drawings and peer review report should be signed by three unlimited licensed structural engineers.

During the course of Authority review, if it is found that the third party consultant or peer reviewer has not fulfilled their obligation to check all the important parameters and aspects of the structural design, Authority will issue a fine for the first three instances. On the fourth instance, the peer reviewer will be suspended from working under Authority's jurisdiction. In order to revoke any such suspension, the third party consultant or peer reviewer will be obliged to submit a new list of qualified engineers, including CV and other relevant documents, to demonstrate their capability.

4.i.4 Summary Report

Drawings of all buildings/projects falling under the abovementioned categories, as outlined in Section 6.1 of this Chapter, should be submitted along with a detailed report and a cover letter expressing satisfaction by the third party consultant or peer reviewer. This shall be stamped and signed by the approved and unlimited licensed structural engineer, then signed by all three unlimited licensed structural engineers of the peer reviewing firm.

The detailed report should include, but not be limited to, the following information:

a. Load summary
b. Wind drift
c. Seismic drift
d. Total drift
e. Displacement drift
f. Inter-storey drift
g. Performance of the tower understrength
h. Serviceability
i. Worst possible loads combinations
j. Other critical information

4.i.5 Special Projects

A joint venture between local consultants and approved/registered international consultants is permitted in the case of special landmarks, prestigious, iconic, complicated projects, subject to the prior approval of Authorities.

4.i.6 Geotechnical Third Party

A geotechnical third party review is required under the same conditions as mentioned under clause 4.i.1. The scope needs to be clearly defined by the project main consultant with the Authorities to ensure all geotechnical requirements are fulfilled and captured into the review process and related detailed third party reports.

4.j Construction Methodology Governing Design

4.j.1 Expansion Joints

Expansion joints on large slab areas should be considered where appropriate based on a cost-benefit analysis. Requirements for additional reinforcement to accommodate self-straining forces needs to be compared to alternative options (such as double structure) before selecting the most appropriate method.

4.j.2 Interface with Adjacent Plots

This detail will need to contain as a minimum the following information

- Waterproofing details
- Pouring sequence
- Slabs heights
- Differential movement allowance and detailing
- Existing earthing works, services outside the plot, etc.

4.j.3 Restraint of compression members

As per the AISC Specification cl. 6.2, AISC Commentary and AISC appendix 6 the requirements for a positional restraint to a compression member are:

- In term of strength, the restraining elements should have the Strength to resist 1.0% of the axial force on the compression member and transfer to adjacent points of positional restraint
- As the floor plate acts as a "bracing system", it is overly conservative to assume that the maximum 1% restraint force is required to be resisted simultaneously by each vertical load carrying element. AISC commentary recommends that when more than one column is being considered out-of-tolerance at the same time, the magnitude of the out-of-plumb is reduced by $1/\sqrt{(n)}$ where n is the number of columns being considered.
- Additional guidance from British Standard 5950-1 cl. 4.7.1.2 indicates that where bracing systems restrain multiple members, it must resist sum of the restraint forces reduced by Kr=sqrt(0.2+1/Nr) where Nr is the number of parallel elements restrained

In terms of service stiffness, bracing must provide a minimum required stiffness of 8*Axial Load / 0.75*Unbraced Length.

For concrete works refer to ACI 117-90: Standard Specifications for Tolerances for Concrete Construction and Materials.

4.j.4 Slab to Core wall Interface

Slip forming is often the choice of formwork system for construction of core walls in tall buildings due to its speed and ease of use. However, a common problem with the use of this system is difficulty on installing the slab to wall detail. The Consultant should consider the use of pull-out bars of size T12 or smaller, where compatible with the floor slab design; reference should be made to the slab design section of this guide. Where possible, use of couplers should be avoided, but where unavoidable requirements for couplers should be identified early in the design process to avoid material price variations late in the project.

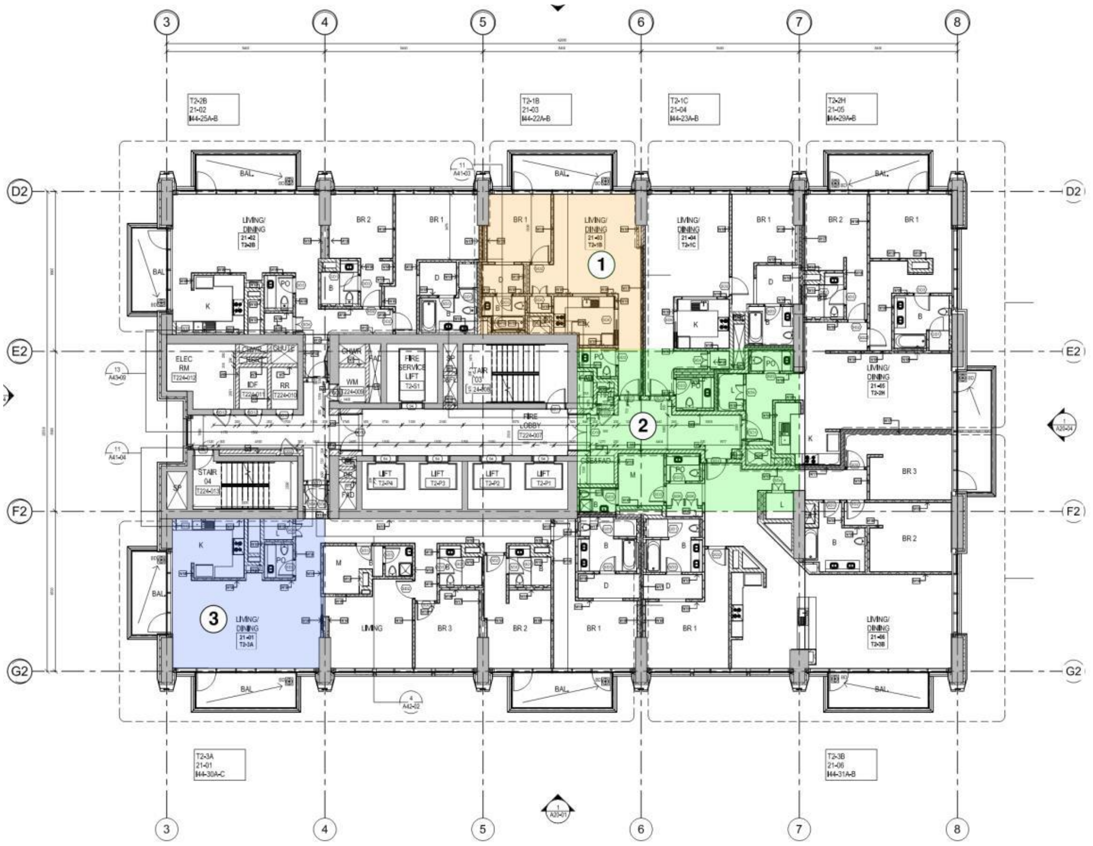
Bars up to 16mm (but preferably smaller than 12mm) in diameter may be used without the need for couplers. A technique known as “bending out” may be used for smaller diameters since these reinforcements can be manually bent on site. Using this technique, the bars dowelling into the slab are pre-bent such that they are concealed inside a shuttering strip. The shuttering casing is flushed into the slip form allowing a seamless movement of the slip form. After casting the wall, the wall forms are removed and the shuttering covering the bent bars are taken out. The bars can then be bent out into position ready to engage the incoming slab reinforcement.

4.k Partition Wall Calculation

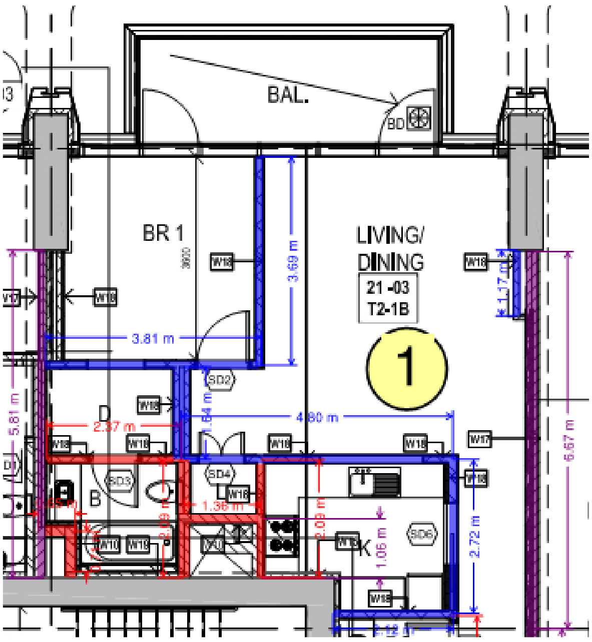
For the purpose of determining equivalent UDL pressure load due to partition walls, following procedure may be updated.

a. Consider at-least three slab panels where architectural layout shows a denser partition wall spacing.
b. Once the sampling bays are selected, the total length of each type of wall is calculated. Openings due to doors and other holes may be ignored for simplicity.
c. Lengths for each type of wall is multiplied with the partition loads and the height of the wall. Height of the wall may be assumed from structural slab to the soffit of the slab above.
d. Consider only 50% contribution from the partition wall located on the slab panel edges.
e. The calculated total wall weight is divided by the area of the bay considered. The result is expressed in kN/m2. This value may then be used for bays with similar wall density. The procedure may be repeated for the other bays.
f. Panels with substantially different partition wall load shall be applied as a separate load.

To illustrate the above procedure, an example is given in the succeeding sections. For this example, three bays are considered to see how the value ranges for different wall densities. The clear height of the walls is taken as the proposed height of 3.30m less the thickness of the slab of 250mm. Resulting clear height is 3.05m.



Bay 1



WALL LEGEND:

- DEMISING WALLS AND WET AREAS (AAC BLOCK WALL AT 1.64kN/m per meter height of wall)
- PARTITION WALL FOR WET AREAS 100mm
- DRY WALL PARTITIONS 100mm

The breakdown of the wall lengths is:

Demising walls = 0.50 (5.81 + 6.67) = 6.24m

Partition wall for Wet Areas = 2.37 + 0.65 + 0.71 + 2.09 + 1.36 + 2.09 = 9.27m

Dry wall partition = 3.81 + 1.64 + 3.69 + 4.80 + 2.72 + 0.5(2.12 + 1.17+1.17) = 18.89m

Floor loads based on the total wall weight may then be calculated:

Weight (Demising walls) = 1.64kN/m2 * 6.24m * 3.05m= 31.21kN

Weight (Wet areas) = 1.06kN/m2 * 9.27m * 3.05m = 30.00kN

Weight (Dry wall) = 0.50 kN/m2 * 18.89m * 3.05m = 28.81kN

Total wall weight on Bay 1 = 31.21 + 30.00 + 28.81 = 90.02kN

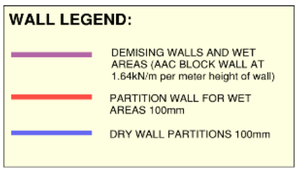
Finally, the floor load based on partitions of Bay 1 is:

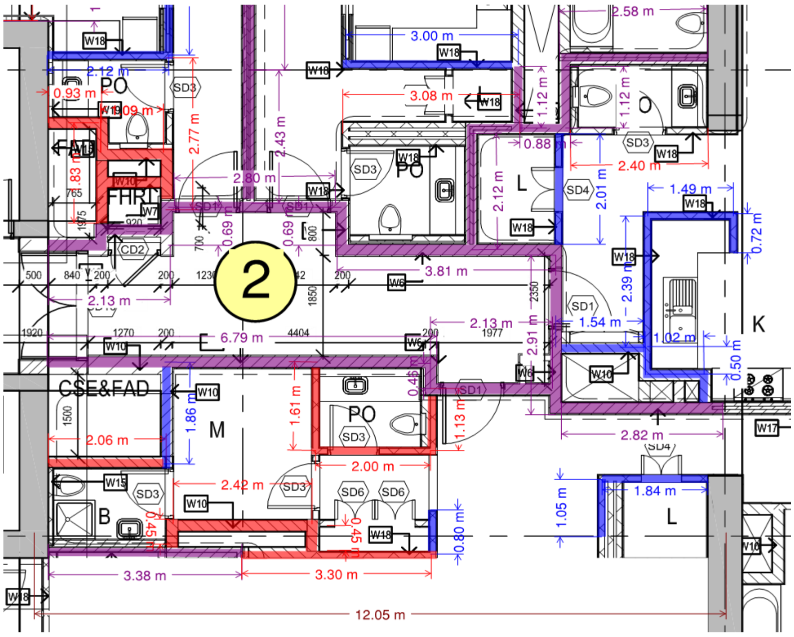
SDL (wall) = Total wall weight on Bay 1 / Floor area of Bay 1

SDL (wall) = 90.02kN / (8.4m * 8.5m)

SDL (wall) = 1.26kN/m2

Bay 2





The breakdown of the wall lengths is:

Demising walls = 2.13 + 0.69 + 0.69 + 2.80 + 2.43 + 2.12 + 1.12 + 0.88 + 1.12 + 3.81 + 6.79 + 2.13+ 0.46 + 2.91 + 2.82 + 0.5(3.38 + 2.58 + 2.12) = 35.25m

Partition wall for Wet Areas = 0.93 + 2*1.09 + 1.83 + 2.77 + 3.08 + 2.40 + 2.06 +2* 0.45 + 1.61 + 2.00 + 1.13 + 0.5(3.30 + 2.42) = 23.75m

Dry wall partition = 1.86 + 2.01 + 1.54 + 2.39 + 1.49 + 0.50 + 1.02 + 1.84 + 1.05 + 0.5(0.72 + 0.80 + 3 + 2.12) = 17.01m

Floor loads based on the total wall weight may then be calculated:

Weight (Demising walls) = 1.64kN/m2 * 35.25m * 3.05m= 176.32kN

Weight (Wet areas) = 1.06kN/m2 * 23.75m * 3.05m = 76.78kN

Weight (Dry wall) = 0.50 kN/m2 * 17.01m * 3.05m = 25.94kN

Total wall weight on Bay 2 = 176.32 + 76.78 + 25.94 = 279.04kN

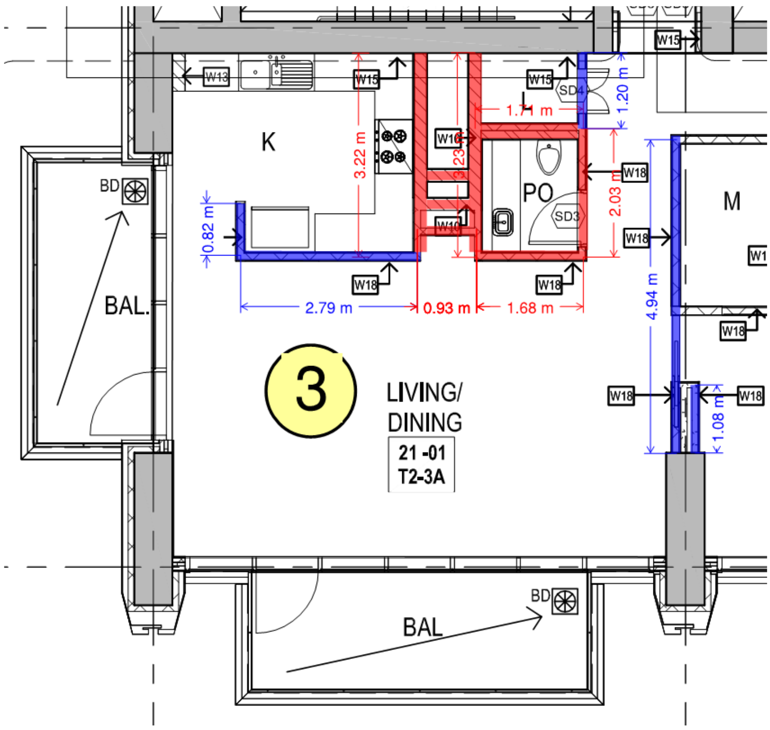
Finally, the floor load based on partitions of Bay 2 is:

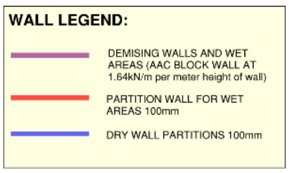
SDL (wall) = Total wall weight on Bay 2 / Floor area of Bay 2

SDL (wall) = 279.04kN / (12.05m * 8.5m)

SDL (wall) = 2.72kN/m2

Bay 3





The breakdown of the wall lengths is:

Demising walls = 0

Partition wall for Wet Areas = 3.22 + 3.23 + 1.71 + 1.68 + 2.03 + 3*0.93 = 14.66m

Dry wall partition = 0.82 + 2.79 + 1.20 + 0.5(4.94 + 1.08) = 7.82m

Floor loads based on the total wall weight may then be calculated:

Weight (Demising walls) = 0
Weight (Wet areas) = 1.06kN/m2 * 14.66m * 3.05m = 47.40kN
Weight (Dry wall) = 0.50 kN/m2 * 7.82m * 3.05m = 11.93kN
Total wall weight on Bay 3 = 0 + 47.40 + 11.93 = 59.33kN

Finally, the floor load based on partitions of Bay 3 is:

SDL (wall) = Total wall weight on Bay 3 / Floor area of Bay 3
SDL (wall) = 59.33kN / (8.4m * 8.5m)
SDL (wall) = 0.83kN/m2 (Use 1.0kPa as per minimum)

4.l Development Classification Criteria (Luxury, Standard, and Economical)

Saftey of structure cannot be a categorizing as well as another discipline for the purpose of Luxury, Economic or moderate design, but there are few points can be considered by another discipline will impact on the cost of the structure. The following bullet points will be a guide to be considered during the concept design by the architectural and MEP.

Cluster	**Luxury**	**Economic**
	Criteria	Criteria
Grid Spacing(Columns, Wall)	Same as explained in Previous sections	Limited to 7m
Finishing thickness	Same as Architectural Schedule but limited to 100mm	Between 60mm to 70mm
Core Wall Arrangement	Highly Recommended to have an aligned core wall to be able coupled	Must have Aligned core wall to be coupled
Partition Type	Can be any type	Must be lightweight/drywall

Plan Configuration	Depends on the concept design of architectural	Tobe the regular shape and follow the recommendation of the arrangement of the structural components which stipulated in the section
Height/ width Ratio (Skinny or Non-Slender)	Depends on the concept design of architectural	To avoid the skinny type of structure it is recommended to be limited be bigger than 8:1