PROFESSIONAL PRACTICE GUIDELINES

BUILDINGS

STRUCTURAL ENGINEERING SERVICES FOR TALL CONCRETE BUILDING PROJECTS

VERSION 1.0
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ENGINEERS &
GEOSCIENTISTS
BRITISH COLUMBIA

ENDORSED BY:

STRUCTURAL ENGINEERS ASSOCIATION OF BRITISH COLUMBIA

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PREFACE

These *Professional Practice Guidelines – Structural Engineering Services for Tall Concrete Building Projects* were developed by Engineers and Geoscientists British Columbia to guide professional practice related to structural engineering services for tall concrete buildings.

These guidelines were first published in 2021 to address unique challenges associated with the design of tall concrete buildings. Topics covered include design for gravity loads, design for lateral wind forces, and design for earthquake ground motions. Much of the guidance focuses on the latter topic, which has changed significantly in recent years. Specifically, this document deals with the seismic design of concrete buildings using Linear Dynamic Analysis, as well as the evaluation of seismic performance using Non-linear Dynamic Analysis, which is increasingly being used for the design of tall concrete buildings.

Most, if not all, tall concrete buildings constructed in British Columbia (BC) are shear wall buildings, and often the shear walls are arranged in a central core. Thus, these guidelines primarily address the requirements of this type of building. But since no minimum height or minimum number of storeys defines the buildings addressed within the scope of this document, many of the concepts and principles for tall concrete buildings apply to low-rise concrete buildings as well. Similarly, much of the guidance provided for tall concrete buildings applies to tall hybrid buildings (i.e., steel or encapsulated mass timber with concrete core walls). The Structural Engineer of Record must use professional judgment to determine whether and how these guidelines apply to a particular building project.

Engineering Professionals are responsible for meeting the requirements of the current edition of the *BC Building Code* or the Vancouver Building By-law (defined collectively in these guidelines from here on as the “Code") and corresponding referenced standards (e.g., CSA A23.3, Design of Concrete Structures). However, information about what is considered good professional practice is evolving more rapidly than the adoption of new editions of the Code, particularly with regard to design for earthquake ground motions. Thus, the *National Building Code of Canada* (*NBC*) 2020 model code and the CSA A23.3:19 standard are referenced throughout these guidelines as considerations where the incoming provisions are more conservative than the requirements of the current edition of the Code and referenced standards (i.e., CSA A23.3:14). (Those documents are expected to be adopted in the next edition of the Code.)

In addition, since there is limited information in Canadian codes and referenced standards regarding the evaluation of seismic performance using Non-linear Dynamic Analysis, when undertaking such analysis registrants are expected to be aware of and consider the following two United States guidelines: *An Alternative Procedure for Seismic Analysis and Design of Tall Buildings in the Los Angeles Region*, published by the Los Angeles Tall Building Structural Design Council (LATBSDC 2020), and the *Guidelines for Performance-Based Seismic Design of Tall Buildings*, published by the Pacific Earthquake Engineering Research Center Tall Buildings Initiative (PEER TBI 2017).

These *Professional Practice Guidelines – Structural Engineering Services for Tall Concrete Building Projects* describe expectations and obligations of professional practice in relation to the specific professional activity of structural engineering services for tall concrete building projects to be followed at the time they were prepared. However, this is a living document that is to be revised and updated as required in the future, to reflect the developing state of practice.

TABLE OF CONTENTS

LIST OF APPENDICES

LIST OF FIGURES

LIST OF TABLES

ABBREVIATIONS

ABBREVIATION	TERM
AHJ	Authority Having Jurisdiction
BC	British Columbia
BCBC	*British Columbia Building Code*
CRP	Coordinating Registered Professional
CSA	Canadian Standards Association
EER	electrical engineer of record
GER	geotechnical engineer of record
GILD	Gravity Induced Lateral Demand
LATBSDC	Los Angeles Tall Buildings Structural Design Council
LFRS	Lateral Force Resisting System
MER	mechanical engineer of record
NBC	*National Building Code of Canada*
PEER TBI	Pacific Earthquake Engineering Center Tall Buildings Initiative
PGA	peak ground acceleration
PSHA	probabilistic seismic hazard analysis
RPR	Registered Professional of Record
SER	Structural Engineer of Record
SEABC	Structural Engineers Association of British Columbia
SFRS	Seismic Force Resisting System
SLS	serviceability limit state
SRP	Supporting Registered Professional
SSRA	site-specific response analysis
ULS	ultimate limit state
UHS	uniform hazard spectrum
VBBL	Vancouver Building By-law

NOTATION

NOTATION	DEFINITION
A	Twisting index
c	Neutral axis depth
C_x	Wall pier compression force, where x is the wall pier indicator
D_{5-95}	Mean significant duration
E	Earthquake load
E_c	Modulus of elasticity of concrete
$E_c I_e$	Effective flexural rigidity
$E_c I_g$	Uncracked section flexural rigidity
f_c'	Concrete strength
f_y	Reinforcement strength
H	Height of the roof
h_s	Storey height
h_w	Height of wall
L	Live load
l_w	Length of wall
M_x	Wall pier bending moment, where x is the wall pier indicator
M_n	Nominal flexural resistance
M_p	Probable flexural resistance
M_r	Factored flexural resistance
MCE_R	Risk-targeted maximum considered earthquake
P_x	Wall pier axial load, where x is the wall pier indicator
$P_{r,max}$	Maximum factored axial load resistance
R_d	Ductility-related force modification factor
R_o	Overstrength-related force modification factor
$R_d R_o / \gamma_{ws}$	Net force reduction factor
S	Snow load
S_a	5% damped spectral response acceleration

NOTATION	DEFINITION
T	Fundamental lateral period of the building
T_1	Fundamental period of the structure based on the effective stiffness of concrete walls given in CSA A23.3 (as a function of elastic bending moment to strength of the wall)
$T_{90\%}$	Lowest period of the modes necessary to achieve 90% mass participation
T_x	Wall pier tension force, where x is the wall pier indicator
T_{min}	Lower bound of period range (minimum period)
T_{max}	Upper bound of period range (maximum period)
T_R	Period range
T_{RS}	Scenario-specific period range
$T_{RS\ Crustal}$	Crustal period range
$T_{RS\ Interface}$	Subduction interface period range
$T_{RS\ Long}$	Long period range
$T_{RS\ Short}$	Short period range
$T_{RS\ Subcrustal}$	Subcrustal (in-slab) period range
$V_{x,x}$	Wall pier shear force, where x indicates the two adjacent wall piers
V_c	Shear resistance – concrete contribution
$V_{r,max}$	Maximum shear resistance
V_S	Shear resistance – steel contribution
V_{s30}	Average shear wave velocity in the top 30 m of soil or rock
α	Severity of a gravity-induced lateral demand
γ_{ws}	Wall overstrength factor for shear
$\delta_{avg,t}$	Average drift ratio
δ_{col}	Column drift ratio
$\delta_{max,t}$ ($\delta_{max,ta}$)	Maximum drift ratio
$\zeta_{critcal}$	Critical damping
θ	Angle from the vertical axis
θ_{id}	Inelastic rotational demand
$\phi_c f'_c$	Factored compression strength of concrete

DEFINED TERMS

The following definitions are specific to these guidelines. These words and terms are capitalized throughout the document.

TERM	DEFINITION
Act	*Professional Governance Act* [SBC 2018], Chapter 47.
Architect	An individual who is registered as an architect by the Architectural Institute of British Columbia under the *Architects Act* [RSBC 1996], Chapter 17, and entitled to practice the profession of architecture in British Columbia.
Authority Having Jurisdiction	The jurisdictional body (usually municipal) with authority to administer and enforce the *British Columbia Building Code*, the City of Vancouver Building By-law, the *National Building Code of Canada* (NBC), or a local building bylaw or code, as well as government agencies that regulate a particular function in a building.
Basis of Design Document	A document prepared by the Structural Engineer of Record for use and approval by a Peer Review panel, for example in the Peer Review of the Non-linear Dynamic Analysis of the building.
Bylaws	The Bylaws of Engineers and Geoscientists BC made under the *Act*.
Capacity Design (Approach)	A methodology of providing a higher capacity against failure due to brittle actions thereby resulting in an overall ductile response of a structure.
Code	The *British Columbia Building Code* (*BCBC*) or the Vancouver Building By-law (VBBL).
Coordinating Registered Professional	A Registered Professional retained under Clause 2.2.7.2.(1)(a) of Division C of the Code to coordinate all design and Field Reviews of the Registered Professionals who are required for a project.
Deformation-Controlled Action/Demand	An action expected to undergo non-linear behavior in response to earthquake shaking, and which is evaluated for its ability to sustain such behavior.
Elastic Deformation	Deformation of a structural member that recovers immediately upon removal of the force that produced it.
Engineering Professional(s)	Professional engineers, professional licensees engineering, and any other individuals registered or licensed by Engineers and Geoscientists BC as a "professional registrant" as defined in Part 1 of the Bylaws.
Engineers and Geoscientists BC	The Association of Professional Engineers and Geoscientists of the Province of British Columbia, also operating as Engineers and Geoscientists BC.
Force-Controlled Action/Demand	An action that is expected to undergo limited non-linear behavior in response to earthquake shaking, and is evaluated based on available strength.
Gravity-Induced Lateral Demand	A constant lateral force applied to the Seismic Force Resisting System by gravity loads.

TERM	DEFINITION
Gravity-Load Resisting Frame	A system of structural members (e.g., slabs, beams, columns, or walls) connected together to transfer gravity loads to the foundation.
Inelastic Deformation	Deformation of a structural member that does not recover upon removal of the force that produced it.
Lateral Force Resisting System	A structural system that transfers lateral forces to the foundation.
Letters of Assurance	Documents set out in a schedule of Subsection 2.2.7. in Part 2 of Division C of the Code used to confirm and assure Code-compliant design and required field reviews by Architects and Engineering Professionals. Otherwise known as Schedules A, B, C-A, and C-B. Refer to the *Guide to the Letters of Assurance in the BC Building Code 2006* (Province of BC 2010).
Linear Analysis	An analysis where the stiffness matrix remains constant.
Linear Dynamic Analysis	A linear analysis of a structure accounting for the movement (acceleration and velocity) of the structure using the modal response spectrum method.
Non-linear Dynamic Analysis	Non-linear evaluation of dynamic response of a structure subjected to a ground motion record. Also commonly referred to as Non-linear Time History Analysis.
Peer Review	The independent evaluation of the work of an Engineering Professional for conceptual and technical soundness by another appropriately qualified Engineering Professional.
Primary Structural System	A combination of structural members that support a building's self-weight and applicable live loads based on occupancy, use of the space, and environmental loads such as wind, snow, and seismic forces. The Primary Structural System comprises the Lateral Force Resisting System and the Gravity-Load Resisting System.
Registered Professional	Defined in the Code as: "a) a person who is registered or licensed to practice as an Architect under the *Architects Act*, or b) a person who is registered or licensed to practice as a professional engineer under the *Engineers and Geoscientists Act*." The *Engineers and Geoscientists Act* has been superseded by the *Professional Governance Act*, which now defines a professional engineer as professional engineers, professional licensees engineering, and any other individuals registered or licensed by Engineers and Geoscientists BC as a "professional registrant" as defined in Part 1 of the Bylaws and having the appropriate scope of practice, all of whom must be qualified by training or experience to provide designs for building projects.
Registered Professional of Record	Defined in the Code as a Registered Professional retained to undertake design work and field reviews in accordance with Subsection 2.2.7. of Division C.
Registrant	Means the same as defined in Schedule 1, section 5 of the *Professional Governance Act*.
Seismic Force Resisting System	The Lateral Force Resisting System designed specifically to resist seismic actions (forces and displacements).

TERM	DEFINITION
Specialty Structural Engineer	An Engineering Professional who designs and supervises the preparation of documents for a specialty structural element while acting as a Supporting Registered Professional providing supplementary supporting structural engineering services to the Structural Engineer of Record.
Struct.Eng.	A specialist designation granted by Engineers and Geoscientists BC to Engineering Professionals who have demonstrated to Engineers and Geoscientists BC that they have the requisite qualifications. Some Authorities Having Jurisdiction stipulate that only a Struct.Eng. can take professional responsibility for structural engineering services on certain types of buildings.
Structural Engineer of Record	An Engineering Professional with general responsibility for the structural integrity of the Primary Structural System. The Structural Engineer of Record takes overall responsibility as the Registered Professional of Record for all items under the structural discipline on Schedule B of the Letters of Assurance in the Code. A Structural Engineer of Record may be required by the Authority Having Jurisdiction to be registered as a Struct.Eng.
Supporting Registered Professional	The Registered Professional providing supplementary supporting design and/or field review services for structural building components, or sub-components, to the Structural Engineer of Record (e.g., secondary structural elements). It is recommended that the Registered Professional of Record obtain and retain in the project files Schedules S-B and S-C from the Supporting Registered Professional in the form provided in Appendix A of the *Joint Professional Practice Guidelines – Professional Design and Field Review By Supporting Registered Professionals* (AIBC and Engineers and Geoscientists BC 2020). These schedules provide assurance confirming that the plans and supporting documents relating to the supporting engineering services for a particular structural component, or subcomponent, substantially comply, in all material respects, with the requirements of the Code.

VERSION HISTORY

VERSION NUMBER	PUBLISHED DATE	DESCRIPTION OF CHANGES
1.0	January 11, 2022	Initial version.

1.0 INTRODUCTION

Engineers and Geoscientists British Columbia is the regulatory and licensing body for the engineering and geoscience professions in British Columbia (BC). To protect the public, Engineers and Geoscientists BC establishes, monitors, and enforces standards for the qualification and practice of its Registrants.

Engineers and Geoscientists BC provides various practice resources to its Registrants to assist them in meeting their professional and ethical obligations under the *Professional Governance Act (*the *Act*) and Engineers and Geoscientists BC Bylaws (Bylaws). Those practice resources include professional practice guidelines, which are produced under the authority of Section 7.3.1 of the Bylaws and are aligned with the Code of Ethics Principle 4.

Each professional practice guideline describes expectations and obligations of professional practice that all Engineering Professionals are expected to have regard for in relation to specific professional activities. Engineers and Geoscientists BC publishes professional practice guidelines on specific professional services or activities where additional guidance is deemed necessary. Professional practice guidelines are written by subject matter experts and reviewed by stakeholders before publication.

Having regard for professional practice guidelines means that Engineering Professionals must follow established and documented procedures to stay informed of, be knowledgeable about, and meet the intent of any professional practice guidelines related to their area of practice. By carefully considering the objectives and intent of a professional practice guideline, an Engineering Professional can then use their professional judgment when applying the guidance to a specific situation. Any deviation from the guidelines must be documented and a rationale provided. Where the guidelines refer to professional obligations specified under the *Act*, the Bylaws, and other regulations/legislation, Engineering Professionals must understand that such obligations are mandatory.

These *Professional Practice Guidelines – Structural Engineering Services for Tall Concrete Building Projects* provide guidance on professional practice for Engineering Professionals who provide structural engineering services for tall concrete building projects.

1.1 PURPOSE OF THESE GUIDELINES

This document provides guidance on professional practice to Engineering Professionals who provide structural engineering services for tall concrete building projects. The purpose of these guidelines is to provide a common approach for carrying out a range of professional activities related to this work.

Following are the specific objectives of these guidelines:

1. Describe expectations and obligations of professional practice that Engineering Professionals are expected to have regard for in relation to the specific professional activity outlined in these guidelines by:
   - specifying tasks and/or services that Engineering Professionals should complete;
   - referring to professional obligations under the *Act*, the Bylaws, and other regulations/legislation, including the primary obligation to protect the safety, health, and welfare of the public and the environment; and
   - describing the established norms of practice in this area.

2. Describe the roles and responsibilities of the various participants/stakeholders involved in these professional activities. The document should assist in delineating the roles and responsibilities of the various participants/stakeholders, which may include the Structural Engineer of Record, Registered Professionals of Record, owners/clients, Authorities Having Jurisdiction, and contractors.
3. Define the skill sets that are consistent with the training and experience required to carry out these professional activities.
4. Provide guidance on the use of assurance documents, so the appropriate considerations have been addressed (both regulatory and technical) for the specific professional activities that were carried out.
5. Provide guidance on how to meet the quality management requirements under the *Act* and the Bylaws when carrying out the professional activities identified in these professional practice guidelines.

1.2 ROLE OF ENGINEERS AND GEOSCIENTISTS BC

These guidelines form part of Engineers and Geoscientists BC's ongoing commitment to maintaining the quality of professional services that Engineering Professionals provide to their clients and the public.

Engineers and Geoscientists BC has the statutory duty to serve and protect the public interest as it relates to the practice of professional engineering, including regulating the conduct of Engineering Professionals. Engineers and Geoscientists BC is responsible for establishing, monitoring, and enforcing the standards of practice, conduct, and competence for Engineering Professionals. One way that Engineers and Geoscientists BC exercises these responsibilities is by publishing and enforcing the use of professional practice guidelines, as per Section 7.3.1 of the Bylaws.

Guidelines are meant to assist Engineering Professionals in meeting their professional obligations. As such, Engineering Professionals are required to be knowledgeable of, competent in, and meet the intent of professional practice guidelines that are relevant to their area of practice.

The writing, review, and publishing process for professional practice guidelines at Engineers and Geoscientists BC is comprehensive. These guidelines were prepared by subject matter experts and reviewed at various stages by a formal review group, and the final draft underwent a thorough consultation process with various advisory groups and divisions of Engineers and Geoscientists BC. These guidelines were then approved by Council and, prior to publication, underwent final editorial and legal reviews.

Engineers and Geoscientists BC supports the principle that appropriate financial, professional, and technical resources should be provided (i.e., by the client and/or the employer) to support Engineering Professionals who are responsible for carrying out professional activities, so they can comply with the professional practice expectations and obligations provided in these guidelines.

These guidelines may be used to assist in the level of service and terms of reference of an agreement between an Engineering Professional and a client.

1.3 INTRODUCTION OF TERMS

There is no established definition of what constitutes a tall building; however, there are several different criteria that may influence whether a building is considered a tall building.

One criterion is the urban context; a 10-storey building surrounded by low-rise buildings would likely be considered a tall building. From a structural engineering perspective, another important criterion is the relative proportions of the building. A building that is not particularly tall in terms of number of storeys may be considered tall if the building is slender, while a

building with significant height and a very large footprint may not be considered tall.

These guidelines were developed for concrete buildings with relatively slender shear walls to resist lateral demands.

See the Defined Terms section at the front of the document for a full list of definitions specific to these guidelines.

1.4 SCOPE AND APPLICABILITY OF THESE GUIDELINES

These guidelines provide guidance on professional practice for Engineering Professionals who carry out structural engineering services for tall concrete building projects. These guidelines are not intended to provide technical or systematic instructions for how to carry out these activities; rather, these guidelines outline considerations to be aware of when carrying out these activities. Engineering Professionals must exercise professional judgment when providing professional services; as such, application of these guidelines will vary depending on the circumstances.

Although these guidelines may provide thresholds above which professional involvement is specified as being required, Engineering Professionals must always use their professional knowledge, experience, and judgment to provide the appropriate level of service that is commensurate with the risk of their professional activities to public safety and/or the environment.

An Engineering Professional's decision not to follow one or more aspects of these guidelines does not necessarily represent a failure to meet professional obligations. For information on how to appropriately depart from the practice guidance within these guidelines, refer to the *Quality Management Guides – Guide to the Standard for the Use of Professional Practice Guidelines* (Engineers and Geoscientists BC 2021a), Section 3.4.2.

This document was written to address the requirements of tall concrete shear wall building projects, and specifically concrete core wall buildings; however, much of the guidance presented in this document applies to low-rise concrete buildings, as well as tall hybrid buildings (e.g., steel or encapsulated mass timber with concrete core walls).

Topics covered include design for gravity loads, design for lateral wind forces, and design for earthquake ground motions. Guidance on the design for earthquake ground motions is the focus of much of this document. Guidance is also provided on the seismic design of concrete buildings using Linear Dynamic Analysis, as well as the evaluation of seismic performance using Non-linear Dynamic Analysis.

While these guidelines were written for Engineering Professionals to use as guidance for professional practice, they may also be useful for others for information purposes, such as consultants, developers, constructors, building officials, and the general public.

1.5 ACKNOWLEDGEMENTS

This document was written by a group of subject matter experts, and was reviewed by a separate group of subject matter experts, stakeholders, and various advisory groups and divisions of Engineers and Geoscientists BC. Authorship and review of these guidelines does not necessarily indicate the individuals and/or their employers endorse everything in these guidelines.

The Structural Engineers Association of British Columbia reviewed these guidelines and provided their official endorsement.

See Appendix A: Authors and Reviewers for a list of contributors.

2.0 ROLES AND RESPONSIBILITIES

2.1 COMMON FORMS OF PROJECT ORGANIZATION

The organization of building projects varies according to the needs of the project and the parties involved. The Structural Engineer of Record (SER) is most commonly in a contractual relationship with either the owner, the Coordinating Registered Professional (CRP) (typically the Architect), or a design-build contractor. Refer to Appendix A of the *Professional Practice Guidelines – Structural Engineering Services for Part 3 Building Projects* (Engineers and Geoscientists BC 2019a) for organizational charts showing both contractual and functional relationships.

Regardless of how the project is organized, the various participants each have particular responsibilities, as described below.

2.1.1 COMMUNICATION AND COORDINATION

Tall concrete buildings have complex structural systems, which are sensitive to the geometry of the building and configuration of the building's structural members. While it is possible to make many buildings with geometrically complex configurations work structurally, by discussing a few structural principles and agreeing upon strategies with the Architect and design team at early stages in the project, the complexity of the structural system—including analysis, detailing, constructability, and, ultimately, performance—can be significantly simplified and/or improved.

Such strategies may involve:

- coordinating the size, location, and geometry of the core(s) (including openings in the walls);
- identifying and/or minimizing irregularities in the structure;
- coordinating the location of gravity-load resisting columns in relation to the core(s);
- coordinating plumbing requirements with the mechanical and electrical Engineering Professionals; and/or
- establishing permissions and limitations for services penetrations through structural members.

2.2 RESPONSIBILITIES

The following sections on roles and responsibilities are not meant to be an exhaustive list of either project participants or their responsibilities. Instead, they outline key considerations that these participants should take when embarking on a tall concrete building project.

2.2.1 OWNER/CLIENT

As discussed in Section 2.1 Common Forms of Project Organization, an owner can also be the SER's client. Regardless of the contractual relationship between the owner and the SER, to ensure the design and construction of the building project meets appropriate standards of public safety and the requirements of the Code, the owner should assume the following responsibilities.

The owner should:

- proceed with a building project only after securing adequate financing, recognizing that a reasonable contingency should be included;
- ensure a CRP or design/build contractor and appropriate Registered Professionals of Record (RPRs) are retained;
- ensure required approvals, licenses, and permits from the Authorities Having Jurisdiction are obtained;

- develop, along with the Architect or the design/build contractor, an appropriate written description of the building project;
- ensure appropriate scopes of work and reasonable schedules of work are developed for RPRs;
- ensure that the need for and cost of independent review and, where applicable, Peer Review, are contemplated and addressed in the contract;
- ensure contracts are finalized with RPRs before their services are required;
- ensure the contracts with RPRs are amended where necessary to include services required beyond the original scopes of work;
- recognize that designs, design drawings, specifications, contract documents, and other documents prepared by RPRs are for that building project only and should not be used or copied for other building projects without consent of the RPRs;
- recognize that some design changes may be required, including those resulting from different interpretations of the Code between the Authority Having Jurisdiction (AHJ) and RPRs; and
- confirm if the SER is to apply the *Professional Practice Guidelines – Sustainability* to the building project (Engineers and Geoscientists BC 2016) and the specific nature of the services to be provided.

If the owner does not assume the above responsibilities, RPRs should:

- consider recommending to the owner in writing that the owner should undertake and perform the responsibilities; or
- consider withdrawing from the building project.

2.2.2 COORDINATING REGISTERED PROFESSIONAL

The role of the CRP, as described in the Letter of Assurance, Schedule A, Confirmation of Commitment By Owner and Coordinating Registered Professional, is to coordinate the design work and field reviews of the RPRs required for the project in order to ascertain that the design will substantially comply with the *British Columbia Building Code* (*BCBC*) or the Vancouver Building By-law (VBBL) (defined collectively in these guidelines as the Code), and other applicable enactments respecting safety.

The role of the CRP is clearly defined in the Code, Note A-2.2.7.2.(1)(a) of Division C.

While the RPRs who provide design and field review services must assume responsibility for their own work, the CRP needs to provide a level of administrative overview beyond simply obtaining authenticated drawings and Letters of Assurance, whether or not the CRP has a contractual relationship with the RPRs involved in the project.

The CRP has certain responsibilities, which may include the following, to enable RPRs to perform their duties appropriately.

The CRP should:

- ensure there is sufficient time permitted for the RPRs to execute their work to the required standards, with sufficient coordination;
- provide timely and appropriately detailed information to allow RPRs to adequately carry out their scope of work;
- coordinate and review designs, specifications, and contract documents prepared by RPRs;
- coordinate communication of information between the owner, the general contractor, and the RPRs, so the building project substantially complies in all material respects with the Code and meets the owner's needs; and
- ensure compliance with the Engineers and Geoscientists BC Bylaw 7.3.5 regarding the completion of documented independent reviews of structural designs.

2.2.3 ARCHITECT

Architectural considerations and responsibilities related to tall concrete buildings include, but are not limited to, the following.

The Architect should:

- interpret the needs of the owner so the designs will meet the intended function of the building project;
- identify and advise RPRs of special design criteria, such as, in the case of the SER, equipment, loads, and span requirements;
- develop the scope of work with RPRs for designs, specifications, contract documents, field reviews, and/or contract administration;
- determine, in consultation with the SER, the most appropriate gravity and lateral system layouts for the building that respond best to the building program (Basis of Design Document);
- determine and advise the SER of the fire-rating requirements of structural members;
- determine, in consultation with the RPRs and Supporting Registered Professionals (SRPs), strategies for utilizing alternative solutions, as appropriate;
- facilitate early-stage coordination of any irregular tall building-related issues, such as proposed alternative solutions, with the AHJ; and
- prepare and manage an integrated model for the coordination of the RPRs work, where applicable.

2.2.4 GEOTECHNICAL ENGINEER OF RECORD

The geotechnical engineer of record (GER) is responsible for the geotechnical aspects of the design for the subgrade support of the building and the associated field reviews. As with any other type of construction, the GER should refer to the *Professional Practice Guidelines – Geotechnical Engineering Services for Building Projects* (Engineers and Geoscientists BC 2020a) and the requirements of the AHJ.

When providing services related to the Non-linear Dynamic Analysis of a tall concrete building, specific considerations and responsibilities include but are not limited to the following.

The GER should:

- determine lateral earth pressure and bearing capacities for gravity and lateral load cases;
- determine the expected foundation settlements based on the loads provide by the SER;
- provide the expected foundation settlements, including differential settlements, to the Architect and the SER; and
- where applicable, conduct a site-specific probabilistic seismic hazard analysis (PSHA) and provide the site-specific response spectrum (i.e., the results) to the SER.

2.2.5 MECHANICAL ENGINEER OF RECORD

The mechanical engineer of record (MER) has overall responsibility for the design and field review of the mechanical systems. As with any other type of construction, the MER should refer to the *Professional Practice Guidelines – Mechanical Engineering Services for Building Projects* (Engineers and Geoscientists BC 2021b) and the requirements of the AHJ. Specific considerations and responsibilities related to tall concrete buildings include but are not limited to the following.

The MER should:

- coordinate services routing and all penetrations, particularly conduits, with the Architect and SER; and
- route the piping and ductwork so that generally, and wherever possible, they do not penetrate members of the Lateral Force Resisting System unless absolutely necessary; in that case, seek approval of the SER and Architect.

2.2.6 ELECTRICAL ENGINEER OF RECORD

The electrical engineer of record (EER) has overall responsibility for the design and field review of the electrical systems. As with any other type of construction, the EER should refer to the *Professional Practice Guidelines – Electrical Engineering Services for Building Projects* (Engineers and Geoscientists BC 2019b) and the requirements of the AHJ. Specific considerations and responsibilities related to tall concrete buildings include but are not limited to the following.

The EER should:

- coordinate the location of the electrical room and conduit routing requirements with the SER; and
- in collaboration with the SER, route the cables, conduits, and raceways so, to the greatest extent possible, they do not penetrate or negatively affect critical members of the Primary Structural System.

2.2.7 STRUCTURAL ENGINEER OF RECORD

The SER has overall responsibility for the design and field review of the Primary Structural System (i.e., the Lateral Force Resisting System and the Gravity-Load Resisting Frame). As with any other type of construction, the SER should refer to the *Professional Practice Guidelines – Structural Engineering Services for Part 3 Building Projects* (Engineers and Geoscientists BC 2019a) and the requirements of the AHJ. Specific considerations for tall concrete buildings include but are not limited to the following.

The SER is responsible for the following:

- Work with the owner, the Architect, or the design/build contractor to develop a scope of work that allows the SER to provide the required designs, specifications, contract documents, field reviews, and/or contract administration, as described in these guidelines and the Code.
- Authenticate the appropriate Code-mandated Letters of Assurance for design and field reviews regarding the designs and supporting documents the SER prepares. This includes taking responsibility for all structural items in the Letter of Assurance, Schedule B, Assurance of Professional Design and Commitment for Field Review, and crossing out and initialing only items that do not apply to the project.
- Review the secondary structural elements, specialty structural elements, or non-structural elements for their impact on the structural design.
  - While the SER may not be directly responsible for the design of secondary structural elements, specialty structural elements, or non-structural elements, the SER is responsible for designing the Primary Structural System to accommodate these other elements, and for allowing for their effects on the Primary Structural System.
  - Such secondary structural elements may include items such as canopies, guardrails, cladding systems and/or glazing support, anchorage of equipment, steel stud partition walls.
- Where the structural design is dependent on the work of SRPs for specialty or secondary structural elements, obtain from the SRP the Schedule S-B Assurance of Professional Design and Commitment for Field Review By Supporting Registered Professional, and Schedule S-C, Assurance of Professional Field Review and Compliance By Supporting Registered Professional. See the *Joint Professional Practice Guidelines – Professional Design and Field Review By Supporting Registered Professionals* (AIBC and Engineers and Geoscientists BC 2020).
- Obtain and coordinate specific loading conditions from other RPRs such as those from landscaping (e.g., planters, trees, artwork), fire trucks or emergency access routes, pools, libraries or fitness rooms, and storage, mechanical, or other specialty rooms.
- Clearly identify all loading requirements on the structural drawings.

- Understand the scopes of other disciplines, in order to better incorporate the SER's design into an integrated and coordinated product.
- Allow for and coordinate mechanical, electrical, and plumbing penetrations, particularly around electrical rooms and near columns and critical structural elements.
- Inform the client of the requirement to engage a wind consultant to perform a wind study, where applicable.
- Ensure sufficient schedule time is available to deliver a design that meets the required standard of practice.
- Clearly identify on the contract documents the expected building (and therefore elevator shaft) drift, and coordinate with the elevating device consulting engineer, where available.
- Ensure a documented independent review of structural designs is completed before the documentation is issued for construction or implementation. See Section 4.1.7 Documented Independent Review of Structural Designs of these guidelines.
- Where applicable, for example for building projects utilizing Non-linear Dynamic Analysis, prepare a Basis of Design Document and ensure a Peer Review of the structural design is completed before the documentation is issued for construction or implementation. See Section 4.3 Peer Review of these guidelines.
- Ensure that the need for and cost of independent review and, where applicable, Peer Review are contemplated and addressed in the contract.
- Be familiar with and, where appropriate, apply the *Professional Practice Guidelines – Sustainability* to the work (Engineers and Geoscientists BC 2016).

2.2.8 SPECIALTY STRUCTURAL ENGINEER OR SUPPORTING REGISTERED PROFESSIONAL

Where a Specialty Structural Engineer is engaged directly by the SER, the Specialty Structural Engineer should work with the SER to clearly develop the Specialty Structural Engineer's scope of work. The Specialty Structural Engineer should also work with the CRP to coordinate with all relevant parties.

Specialty Structural Engineers are responsible for the integrity of their designs and must authenticate the documents prepared in their professional capacity or under their direct supervision. See Section 4.0 Quality Management in Professional Practice.

Because the Specialty Structural Engineer acts as an SRP, providing supporting engineering services to the SER, the Specialty Structural Engineer must submit to the SER an authenticated Schedule S-B, Assurance of Professional Design and Commitment for Field Review By Supporting Registered Professional, and Schedule S-C, Assurance of Professional Field Review and Compliance By Supporting Registered Professional, as identified in the *Joint Professional Practice Guidelines – Professional Design and Field Review By Supporting Registered Professionals* (AIBC and Engineers and Geoscientists BC 2020).

2.2.9 DESIGN-BUILD CONTRACTOR

For design/build projects, the CRP would typically be a representative of the design/build contractor, and the design/build contractor would typically be contractually obligated to ensure that the CRP adequately discharges the responsibilities of a CRP.

The CRP must still fulfill the expectations and obligations of their professional practice.

2.2.10 GENERAL CONTRACTOR

A general contractor has a contractual relationship with an owner. This contract typically states that the general contractor is responsible for the labour, materials, and equipment for the building project, as well as the construction methods, techniques, sequences, procedures, safety precautions, and programs associated with the construction, as set out in the contract documents.

The general contractor is responsible for the general contractor's own work, the supervision and coordination of the subcontractors' work, and the inspection of the subcontractors' work prior to field reviews by the SER and by the SRP, where applicable. The general contractor is responsible for providing reasonable notice to the SER and the SRP when components are ready for field review.

The fact that field reviews and compliance inspections are conducted by others does not absolve the general contractor of the responsibility to execute works in accordance with construction documents. The general contractor must provide independent quality control.

2.2.11 AUTHORITY HAVING JURISDICTION

An AHJ is responsible for enforcing the Code, policies, standards, and bylaws, or for assessing compliance with the Code, standards, and local bylaws. AHJs can be provincial, municipal, townships, districts, First Nations, or other organizations such as Technical Safety BC.

The AHJ receives authenticated permit submissions, including Letters of Assurance, from the CRP or the RPRs at appropriate times during the building project. The AHJ should confirm that the submissions have been properly completed; and if deficiencies are identified, clearly communicate to the CRP and/or RPRs the specific items that require further attention.

An AHJ may perform inspections as part of its compliance assessment. Inspections by the AHJ do not eliminate the requirement for Engineering Professionals to conduct field reviews of their scopes of work.

3.0 GUIDELINES FOR PROFESSIONAL PRACTICE

3.1 OVERVIEW

These guidelines for the design of tall concrete buildings are presented in three separate sections.

- Section 3.2 Design for Gravity Loads includes:
  - considerations for estimating applied loads in tall concrete buildings;
  - guidance for the design of columns and bearing walls, floor slabs, transfer girders and transfer slabs, and foundations; and
  - miscellaneous considerations for other building elements, such as elevators.
- Section 3.3 Design for Lateral Wind Forces includes:
  - considerations for determining wind forces;
  - serviceability limit state (SLS) and ultimate limit state (ULS)criteria;
  - modelling considerations;
  - guidance for the strength design of the Lateral Force Resisting System (LFRS) for wind forces; and
  - considerations for the use of supplementary damping systems.
- Section 3.4 Design for Earthquake Ground Motions includes:
  - considerations for preliminary design;
  - guidance for determining seismic demands using Linear Dynamic Analysis;
  - considerations for design of concrete shear wall cores;
  - guidance for refined analysis of structure below plastic hinge zone;
  - considerations for design of Gravity-Load Resisting Frames for seismic deformation demands;
  - advanced design issues, such as addressing irregularities; and
  - evaluating life safety performance using Non-linear Dynamic Analysis.

This document outlines the services a Structural Engineer of Record (SER) should provide for a tall concrete building project, and may help an SER explain services to a client, whether the client is an owner, an Architect, or a design/build contractor.

These outlines are not intended to be exhaustive, and do not detract from other provisions of these guidelines.

3.1.1 CONSIDERATION OF RISK

The Engineering Professional has a professional responsibility to uphold the principles outlined in the Engineers and Geoscientists BC Code of Ethics, including protection of public safety and the environment. As such, the Engineering Professional must use a documented approach to identify, assess, and mitigate risks that may impact public safety or the environment when providing professional services.

One of the risk factors that must be considered is climate change implications on the building. Engineering Professionals have a responsibility to notify their clients of future climate-related risks, reasonable adaptations to lessen the impact of those risks, and the potential impacts should a client refuse to implement the recommended adaptations. Engineering Professionals are themselves responsible

for being aware of and meeting the intent of any climate change requirements imposed by a client or Authority Having Jurisdiction.

Construction and operation of buildings contribute significantly to global CO_2 emissions, and the manufacturing of Portland cement, in particular, results in significant CO_2 emissions. These guidelines are not intended to advocate for tall concrete buildings or for concrete as a building material, but instead are intended to describe expectations and obligations of Engineering Professionals who provide structural engineering services for tall concrete buildings.

In considering climate change implications for tall concrete building projects, the SER, in collaboration and communication with the owner and other Engineering Professionals, should consider the environmental impact of tall concrete buildings and opportunities to mitigate the impact. When assessing the appropriateness of any material substitutions, the SER must also consider the structural performance; such considerations should include, but not be limited to, whether the proposed substitution is codified or if an alternative solution would be required, the availability of research and technical guidance, and the general limitations of use.

Other areas of risk encountered in professional practice are quality, technical, financial, and commercial risks. Engineering Professionals should consider risks in such areas using techniques that are appropriate to their area of practice.

3.1.2 APPLICABLE CODES AND STANDARDS

Codes are regularly revised as knowledge and experience progress and new technologies are developed. Model codes are developed at the national level, then adopted at the provincial level, and finally enforced at the local government level. As such, there is often a delay between when research is conducted and new technologies are developed and published, informing the industry of future expectations and good professional practice, and when that information is adopted into the requirements of the *BC Building Code* (*BCBC*) or the Vancouver Building By-law.

It is the Engineering Professional's responsibility to meet the requirements of the Code and standards currently in force. As such, the version of the Code and/or standard is generally not referenced in these guidelines unless the difference between versions is being highlighted or versions not yet in force are being referenced. Codes and standards referenced without the version should be read as those currently in force. More recent (i.e., not yet adopted) versions of codes and standards typically are also Code-compliant; however, if they are not, and the Engineering Professional chooses to apply the impending requirements, an alternative solution would be required.

At the time of publication, the following codes related to structural engineering services for tall concrete building projects (defined collectively in these guidelines as the "Code") were in force:

- *British Columbia Building Code* (*BCBC*) 2018
- Vancouver Building By-law (VBBL) 2019

As well, at the time of publication, the following standard related to structural engineering services for tall concrete building projects was in force:

- CSA A23.3-14, Design of Concrete Structures (referred to from here on as CSA A23.3)

The CSA A23.3:19, Design of Concrete Structures standard and the model code, *National Building Code of Canada 2020* (referred to from here on as *NBC 2020*), are referenced throughout these guidelines as considerations where the incoming provisions are more conservative than the requirements of the Code and referenced standards currently in force. These documents are expected to be adopted in the next edition of the Code. The model code, *National Building Code of Canada 2015* (referred to from here on as *NBC 2015*), is also referred to when important information is provided in the Structural Commentaries to the *NBC 2015* and when the requirements differ from the Code and/or *NBC 2020*.

In some cases, specifically where guidance is not provided in current or incoming editions of the Code, the expectations for professional practice are based on consensus summary combining the research and expertise of Engineering Professionals who are subject matter experts on the life safety performance evaluation of tall concrete buildings in BC with the requirements of the following two United States guidelines:

- *An Alternative Procedure for Seismic Analysis and Design of Tall Buildings in the Los Angeles Region*, published by the Los Angeles Tall Buildings Structural Design Council, referred to in this document as the "LATBSDC guidelines" (LATBSDC 2020)
- *Guidelines for Performance-Based Seismic Design of Tall Buildings*, published by the Pacific Earthquake Engineering Center Tall Buildings Initiative, referred to in this document as the "PEER TBI guidelines" (PEER TBI 2017)

Specific requirements for parking garages are outside the scope of these guidelines. The SER should note that the requirements of CSA S413-14, Parking Structures, and any other Code-referenced standards that apply to the project, must be met.

3.2 DESIGN FOR GRAVITY LOADS

3.2.1 INTRODUCTION AND SECTION OVERVIEW

This section provides considerations for the design of tall concrete buildings for gravity loads, including estimating applied loads, and is broken down into the following sections:

- Section 3.2.2 Columns and Bearing Walls
- Section 3.2.3 Floor Slabs
- Section 3.2.4 Transfer Girders and Transfer Slabs
- Section 3.2.5 Foundations
- Section 3.2.6 Miscellaneous Considerations

3.2.2 COLUMNS AND BEARING WALLS

3.2.2.1 Introduction

Columns and bearing walls are critical components in the Gravity-Load Resisting Frame of tall concrete buildings. The failure mode of columns or bearing walls in tall concrete buildings can be very brittle (resulting in failure with little or no warning) and consequences of failure are usually severe. As a result, the Code requires a very low probability of failure for a column or bearing wall.

The SER should be cautious about relying on experience from previous projects to design columns or bearing walls, as there is no feedback on what probability of collapse was actually achieved in their previous designs.

3.2.2.2 Estimating Applied Loads

The SER must make an accurate estimate of all loads applied to columns and bearing walls in tall concrete buildings.

With concrete structures, the dead load due to the self-weight of the building is a very large portion of the load; therefore, the estimate of the self-weight of the building for the gravity-load design of columns and bearing walls must be equal to or larger than the actual self-weight. However, an overestimate of gravity loads may be unsafe for seismic design; this is discussed in

Section 3.4.4.1 Element Design Forces. The SER should provide allowances and tolerances for the self-weight of the structure over which the SER has control; tighter control of concrete dimensions on the structural drawings and during field reviews permits smaller allowances.

With tall concrete buildings, any error made per floor can accumulate significantly over the height of the building. The SER should allow for the additional weight of the levelling grout applied to the floors, as this can result in a significant increase in the dead load on a column that supports numerous floors.

One way the SER can verify that the estimates of gravity load used in design are consistent with what actually is built is to include the information on the structural drawings. The Code requires that structural drawings include sufficient detail to allow for the determination of dead loads, as well as show all other loads that were used in the design of both the structural members and the exterior cladding. One means by which this requirement can be satisfied is to provide a table of design loads on the structural drawings. However, this approach can leave uncertainty as to the specific location in the building where these design loads were applied in the structural design.

The SER should consider indicating explicitly, on the floor or roof plans, the design loading used for the design of that floor/roof and supporting structure. For floors or roofs with multiple occupancies or significant variations in loading intensities (e.g., landscaped areas or heavy cladding systems), the SER should consider providing a separate plan drawing (i.e., a loading key plan) indicating the design loading assumptions for even greater clarity. This information can be very useful to all who reference the structural drawings; current and future Architects and Engineering Professionals can rely on this information for review and coordination of the structure, and building owners can rely on this information for maintenance and operations. This information is also useful to the contractor during construction to be able to verify that concrete or landscaping thicknesses, for example, are not exceeded.

The *Professional Practice Guidelines – Structural Engineering Services for Part 3 Building Projects*, Section 3.3.3 Building Permitting Stage includes a summary of the minimum information that structural drawings should show for a building permit (Engineers and Geoscientists BC 2019a).

Following are examples of loads that must be properly accounted for:

- Floor-levelling grout: the control of floor flatness during construction and slab deflections significantly influences the amount of floor levelling material to be added, so the SER should allow for 0.25 to 0.5 kPa or more.
- Floor finishes in luxury units; for example, heavy stone.
- Floor finishes on balconies; for example, heavy tiles, sloped topping.
- Concrete topping used to provide slopes on roofs and terraces, due to the potential for a significant amount of concrete to be added.
- Concrete housekeeping pads for mechanical and electrical equipment.
- Elevator machinery.
- Ceilings and the electrical and mechanical services in the ceilings.
- Movable partition walls: steel stud and gypsum board walls typically fall within the Code-prescribed allowance of 1.0 kPa.
- Fixed partition walls: concrete masonry walls can be particularly heavy, so they should be accounted for specifically and shown on the key plan.
- Exterior cladding, window walls, curtain walls, precast elements, brick, and similar elements.
- Window washing equipment and fall-arrest systems.
- Mechanical units on the roof.
- Mechanical screens.

3.2.2.3 Estimating Load Distribution to Columns and Bearing Walls

Reinforced concrete floor systems (i.e., slabs and beams) are highly indeterminate with multiple load paths. They normally have considerable ductility that allows for redistribution of any overload. The same cannot be said about the vertical supports for the floor systems (i.e., the columns and bearing walls). Whereas concrete slabs typically deflect significantly or show other signs of overload prior to failure, columns and bearing walls may fail in a brittle manner with very little prior deformation.

The distribution of gravity loads from floor systems to columns and walls is a complex phenomenon that is influenced by such things as the construction sequence, creep and shrinkage of concrete, and the changing stiffness of members as the load is increased. More complex models do not necessarily result in a more accurate estimate of the force distribution, because they are not able to accurately account for all the factors. The SER must pay special attention to making a safe estimate of the design forces on columns and bearing walls.

Pattern loading for live loads are typically not required for residential floors (per CSA A23.3, Clause 13.8.4) but may be warranted for floors with higher live loads, such as retail floors, amenity floors, plazas, and floors with heavy mechanical and electrical equipment. The SER should consider pattern loading effects where uneven distribution of loads or finishes, including landscaping, could result in large variations in loading effects. Pattern loading is a particularly important consideration for the design of transfer slabs, multi-spans, or cantilever transfer beams.

Whenever an analysis problem is complex and sensitive to the assumptions, the SER should consider a number of different solutions in order to determine the range of effects; this is often referred to as “bounding the solution.” The following are possible approaches to determining column design loads:

- Floor-by-floor analysis using simple tributary areas: this solution should always be considered.
- Three-dimensional Linear Analysis of the floor system: this solution provides a different force distribution that depends on the assumed stiffnesses, which are constantly changing as a floor system is loaded.
- Full three-dimensional analysis of the entire building.

An added complexity in the full building model is how the axial stiffness of the columns and bearing walls is modelled. The axial stiffnesses of these members depends on the in-situ concrete properties, the level of creep deformations and shrinkage that have occurred previously, and the rate at which the members are loaded to the design force level. When the axial deformation of the supporting columns is of similar magnitude as the deflection of the transfer girder or slab, the SER should consider a range of stiffness values. It is possible to use a full three-dimensional model and account for construction sequence and long-term effects (e.g., creep and shrinkage); however, the uncertainty increases with the complexity of the model.

When some of the columns and bearing walls supporting a floor/roof level are themselves supported on transfer girders or transfer slabs, the analysis of the level in question becomes more complex since the stiffness of the transfer elements on the level below are difficult to estimate. Diagonal cracking of these transfer members can dramatically influence the force distribution. The SER should pay special attention when determining the design forces for columns and bearing walls supported on transfer elements, and for columns and bearing walls supporting transfer elements.

Special attention also needs to be paid to increased demands on the supporting elements of transfers due to vertical earthquake motions; this is discussed further in Section 3.4.7.3 Discontinuous Elements Supporting Gravity Loads.

3.2.2.4 Determining Column and Bearing Wall Resistance

The lateral deformations of a building due to earthquake ground motions and wind forces will reduce the axial resistance of columns and bearing walls. The reduction in axial resistance depends on the dimension of the member perpendicular to the axis of bending. A bearing wall, for example, will have a very large reduction in axial resistance when the building deforms in the direction of strong-axis bending of the wall. For this reason, the SER should design the member to have some reserve axial resistance, while keeping in mind that all walls, including so-called bearing walls, will attract seismic actions. Considerations for determining wind-induced and seismic-induced demands are provided in Section 3.3 Design for Lateral Wind Forces and Section 3.4 Design for Earthquake Ground Motions, respectively.

CSA A23.3, Clause 14.2.3 requires that the factored resistance of bearing walls must account for strong axis bending due to the resultant of the axial load not being at the centroid of the wall, or due to lateral deformation of the building caused by wind or seismic demands.

Second-order bending of columns and bearing walls must be properly accounted for when determining the resistance of columns and bearing walls. As a minimum, all compression members should be designed to withstand the total axial load applied at the minimum eccentricity (15 mm + 0.03 times the overall thickness of the member), assumed to be uniform over the full height of the member (i.e., bent in single curvature).

Thin Walls

Thin reinforced concrete walls can be a cost-effective structural element, provided the loads applied to the member are small. Research has demonstrated that heavily loaded thin concrete walls are particularly brittle and can lose all axial load-carrying capacity with little or no warning (Adebar 2013). As a result, CSA A23.3-14, Clause 10.10.4 was revised to require reducing the maximum factored axial load resistance of compression members that are less than 300 mm in dimension, and for walls that are not tied along the full length.

Thin concrete walls should not be used to resist significant gravity loads in tall concrete buildings, particularly in high seismic regions where the walls could be damaged by lateral deformation of the building due to earthquake motions. If the interstorey drift ratio at any point in a building exceeds 0.5%, all walls in the entire building that are assumed to support gravity loads must contain two layers of uniformly distributed reinforcement with a minimum clear spacing of 50 mm between layers (CSA A23.3, Clause 21.11.3.3.1).

Column Offsets

A column offset occurs when the outline of the column above does not fall within the outline of the column below. CSA A23.3 does not include provisions on how to design a column offset.

The increased bearing stresses that occur at the interface of the two offset columns are only a small part of the design problem. The change in location of the resultant axial compression in the column above and below the offset generates a bending moment that is usually resisted by the core. The concrete diaphragms at the level of the offset, as well as the diaphragms above and below the offset, transmit the associated shear forces to the core. When the bending moment from a column offset is resisted by the core, special attention is required in the seismic design of the core to account for the Gravity-Induced Lateral Demand (GILD). This is discussed further in Section 3.4.7.1 Gravity-Induced Lateral Demand Irregularity.

Careful detailing is required in all affected members in the load path between the column offset and the foundation. For example, the columns above and below the offset require additional crack control reinforcement. Special attention is needed when large forces are being transferred from the columns to the core by passive reinforcement rather than by active prestressing force in the slabs.

Fire

The tragedy of the Grenfell Tower fire in 2017 increased the awareness of fire safety in tall buildings. Concrete is inherently more resistant to fire than other construction materials, but it has limitations. The main consideration is providing sufficient thickness of concrete and sufficient concrete cover to the reinforcement to protect the reinforcement from losing strength.

The Code requires a minimum 2-hour fire-resistance rating in many situations; however, in some instances, a more stringent requirement may be required, and local jurisdictions may specify requirements in excess of those specified in the Code. It is the responsibility of the Architect to determine the minimum required fire-resistance rating of the structure.

All columns and walls must be designed for a minimum fire-resistance rating. Typically, the Code requires that the fire-resistance rating of load-bearing walls and columns be greater than or equal to that of the assembly being supported. That is, the columns and walls supporting a floor slab must have at least the fire-resistance rating required for the floor slab. Columns must satisfy minimum dimensions for the applicable fire rating; where small columns are used, Appendix D of the *NBC 2020*, Clause D-2.8.2 prescribes a factor by which the columns must be overdesigned.

In most cases, all sides of the column should be considered exposed to fire; however, in the case of columns embedded in partition walls that are fire separations between units, it would be reasonable to assume that a column is exposed to fire on only the exposed sides. This approach is allowed by CSA A23.3, Clause D-2.8.6 for columns built into a masonry or concrete wall; while CSA A23.3 does not mention drywall partitions for this application, it may be appropriate to consider those materials as well. All fire protection assumptions should be confirmed with the Architect and fire protection engineer, when retained, and must be clearly stated on the structural drawings for future reference by Architects or Engineering Professionals involved in renovations.

The SER should consider relying on concrete cover for the fire protection of columns rather than gypsum board, as there is no way to ensure that the gypsum board would be properly installed and remain in place throughout the life of the building.

3.2.2.5 Influence of Column Design on Other Components

When columns are closely spaced—for example, 3 m apart or less for typical slab thicknesses—the columns and interconnecting slabs may act as a frame that resists lateral loads. Similarly, if columns are located close to the core—for example, within 6 m for typical slab thicknesses—the columns, core, and interconnecting slabs will act as a lateral load resisting frame. See Section 3.4.2 Preliminary Design Considerations (for earthquake design) for further discussion.

The layout of columns supporting the floor systems may have a significant influence on the unbalanced over-turning moment applied to the LFRS (i.e., the core). This is particularly a concern in the coupled wall direction. The unbalanced bending moments applied to the core generate a GILD on the core. This is discussed further in Section 3.4.7.1 Gravity-Induced Lateral Demand Irregularity.

The differential vertical movement of columns and walls due to differential creep deformations may have a significant impact on the floor systems, and is discussed in Section 3.2.3.3 Flexural Design of Slabs.

The SER should also consider thermal effects on exterior columns and differential movement. Differential temperature effects may require columns to be kept primarily within the building envelope for tall concrete buildings.

3.2.3 FLOOR SLABS

3.2.3.1 Estimating Applied Loads

Much of the discussion in Section 3.2.2.2 Estimating Applied Loads (for columns and bearing walls) also applies to estimating loads on floor slabs. A number of additional issues specifically related to floor slabs are presented here.

The Code-specified live loads are a minimum and may not be sufficient for all conditions. The SER must account for the actual live load expected for the building. Examples of common scenarios where the Code-specified live loads may not be sufficient for tall concrete buildings include, but are not limited to:

- 3.6 kPa for mechanical rooms, where particularly heavy or closely spaced equipment is located;
- 2.4 kPa for offices with high-density filing systems, where loading may be closer to that of a library; and
- 4.8 kPa for assembly loading on suspended ground floor slabs, because many tall buildings require firetruck access (for which maximum loads vary by AHJ) at the ground floor level.

The significant weight of landscaping also needs to be accounted for, with the appropriate ultimate limit state (ULS) load factor applied for the given soil depth.

For tall buildings, window-washing systems and fall-restraint systems designed by Specialty Structural Engineers can apply significant localized forces on floors, and those systems have special connection requirements for connections of davit arm bases or fall-restraint anchor posts where they fasten to slabs.

The SER should be aware that the *NBC 2020* has changed how the live load reduction is applied to the design of slabs.

3.2.3.2 Punching Shear

Concrete slabs can tolerate enormous overload without failing in flexure. The critical failure mode for two-way slabs (without drop panels or column capitals) is often punching shear around the column. Punching shear failure is very brittle and sudden, and thus a safe design is critical. In addition, some reserve punching shear strength in the slab is needed to resist any unbalanced moment transfer resulting from the lateral deformations of the building.

Large lateral deformations of buildings due to earthquake ground motions cause flexural cracking around the column, which reduces the factor of safety against punching shear failure. The SER must conduct a specific check for the reduced punching shear capacity due to seismic deformations. See Section 3.4.6 Design of Gravity-Load Resisting Frames for Seismic Deformation Demands. When the interstorey drift ratios are large, transverse reinforcement may be needed in the slab to prevent a punching shear failure.

Sleeves, Ducts, and Conduits

The SER should account for the effect of sleeves, in-slab ducts, and conduits when calculating punching shear resistance.

Since it is very likely that sleeves will be added just before the concrete is placed, the SER should be involved in determining how many and where sleeves can be located. For columns in parking levels, it is prudent to allow for at least one hole in the short dimension of the columns for drainpipes. For columns in parking garages, it is recommended to allow for at least one 150-mm-diameter round hole on the narrow end of the columns on the opposite end from the maneuvering aisle (to accommodate drain lines, which are generally routed down the “back” of the column to prevent car impact).

In-slab ducts located near columns are a concern as they may cause a punching shear failure of the slab, especially in highly stressed slabs such as transfer slabs. Piping and manifolds should be kept out of the critical punching shear zone around the column.

The SER should consider specifying a thicker slab near electrical rooms to allow for conduit congestion.

Structural Integrity Reinforcement

If a punching shear failure occurred in a slab, and the weight of the slab dropped onto the slab below, a progressive failure could potentially result in the complete collapse of the building. Thus, CSA A23.3 requires that structural integrity reinforcement be provided to support the weight of the slab after a punching shear failure.

This reinforcement should consist of at least two reinforcing bars or two prestressing tendons that extend through the column in each span direction. Integrity reinforcement is not required if there are beams containing shear reinforcement in all spans framing into the column.

3.2.3.3 Flexural Design of Slabs

Slab thickness is determined to control deflections (serviceability limit state, or SLS), while the quantity of flexural reinforcement is determined to provide the required flexural strength (ULS). The quantity and distribution of flexural reinforcement will also influence the cracking of slabs (SLS).

Slab Deflections

Table 9.2 in CSA A23.3 provides guidance for the span-to-depth ratios of slabs. That table was developed for slabs supported by columns and walls in a regular arrangement; the SER should exercise caution and conservatism for all slabs and particularly ones with an irregular arrangement of supports. It is recommended that the SER supplement the tabulated span-to-depth ratios with calculations to determine the expected deflections of the slab. Over time, with calculations and field observations of performance, the SER can confirm or modify the span-to-depth ratios used for design.

The continuity (negative) bending moments at the support, which are strongly influenced by the adjacent spans, have a pronounced effect on the deflections of slabs. As a result, in order to effectively control deflection, exterior (end) spans should generally be shorter than interior spans for the same thickness of slab.

The long-term deflections along the exterior façade of the building must be limited because the envelope system (window wall or curtain wall) has a limited displacement tolerance (typically, up to 19 mm deflection is allowed by window wall manufacturers). The cladding system may be damaged if the differential deflection of the slab after the finishes are installed exceeds the tolerance for the cladding system. This can be a very costly repair on tall concrete buildings.

A variety of different methods can be used to estimate the deflection of a concrete floor system. One method is to use a simple two-dimensional frame analysis (strip) program. Another approach is to use a commercially available linear finite element analysis program. A combined approach is to conduct a finite element analysis to determine the immediate deflections, and to estimate the long-term deflection using the simplified procedures in CSA A23.3. In any case, the SER must understand the input and output of the deflection calculations (i.e., the SER does not rely on a “black box” for the design), and must check and calibrate assumptions and design decisions based on multiple field observations.

Factors that influence slab deflections include:

- age and compressive strength of the concrete at the time the forms are removed;
- sequence used for removing forms and installing reshores;
- location of reshores in the span (note this typically cannot be considered in design as the shoring system is unknown);
- number of levels of reshores;
- amount and arrangement of reinforcement;
- amount of shrinkage; and
- actual loading on the slab.

Refer to the *Professional Practice Guidelines – Professional Engineering Services for Temporary Structures: Formwork, Falsework, and Reshore* (Engineers and Geoscientists BC 2021c) for more information.

Floor Flatness and Levelness

Non-level or non-flat floors, including those built within allowable construction tolerances, can accentuate slab deflection issues.

Differential vertical displacements of columns and walls supporting slabs can contribute to a floor not being level after the creep deflections occur. The long-term vertical displacements of columns and walls is dependent on the magnitude of the axial compression stress. Columns with high axial compression stress will have larger creep deflections than columns with low axial compression stress, or than walls, which typically have a lower compression stress inherently.

This differential vertical movement of columns and walls can have significant impact on the levelness of floor slabs. It is possible for the slab formwork to be adjusted to correct for this differential movement; however, this is not a common practice so should be discussed and coordinated with the contractor prior to specifying.

3.2.4 TRANSFER GIRDERS AND TRANSFER SLABS

Transfer girders or transfer slabs are used to transfer the gravity loads from one or more columns or bearing walls at the level that the columns or bearing walls are discontinued. Seismic Force Resisting Systems (SFRSs) that resist lateral loads due to earthquake must be continuous down to the foundation and cannot be supported on transfers, as per Sentence 4.1.8.10.(3) of the Code. When a column location shifts by a sufficiently small amount, a column offset may be used in place of a transfer element. See Section 3.2.2.4 Determining Column and Bearing Wall Resistance.

Transfer girders and transfer slabs complicate the estimate of the load distribution to the columns and bearing walls, as discussed in Section 3.2.2.3 Estimating Load Distribution to Columns and Bearing Walls.

Transfer girders and transfer slabs are often deep members. Sometimes the lower portion of the member is cast first and is used to support the weight of the upper portion. When a pour joint exists within a member, careful consideration should be given to the preparation of the interface surface and the resulting resistance to horizontal shear flow along the surface. Additional longitudinal and transverse shear reinforcement may be required in the lower portion of the member if it is to be used to support the weight of the fresh concrete when casting the top portion of the member.

Transfer girders and transfer slabs are often shear-critical members. When a deep transfer element is supported on a narrow support such as a wall, special attention needs to be paid to the anchorage of the longitudinal reinforcement. At the face of the support, the longitudinal reinforcement must be capable of resisting the shear demand on the longitudinal reinforcement (see CSA A23.3, Clause 11.3.9.5).

Special attention also needs to be paid to the anchorage of the transverse shear reinforcement in deep transfer elements. Although transverse shear reinforcing bars with 90-degree hooks at the bottom end have been commonly used as stirrups in transfer slabs and mat foundations, these do not meet the CSA A23.3 anchorage requirements because the cover cannot be considered restrained against spalling. Adding terminators that form mechanical anchorage at the bottom end of the reinforcing bars can be considered a Code-compliant solution.

Significant challenges occur when transfer girders or transfer slabs frame into the core. The interstorey drift of the building due to earthquake ground motions may induce a large bending moment in the transfer element, and a large axial compression in the member(s) supporting the other end of the transfer girder or transfer slab. See Section 3.4.6 Design of Gravity-Load Resisting Frames for Seismic Deformation Demands.

3.2.5 FOUNDATIONS

Foundation settlements, particularly differential settlement, must be accounted for in the structural design of tall concrete buildings.

Differential settlements are of particular concern as they may result in a magnified horizontal displacement at the top of the building, due to the ratio of building height to foundation width.

The SER should provide the geotechnical engineer of record (GER) with the gravity loads, including distribution that the building will apply to the soil, so the GER can determine—and report to both the Architect and SER—the expected total and differential foundation settlement.

3.2.6 MISCELLANEOUS CONSIDERATIONS

3.2.6.1 Roof-Mounted Systems

Tall buildings require systems to facilitate the exterior envelope maintenance for the building (i.e., window washing). Such systems may include fall-restraint anchors or crane and gantry systems to support bosun chairs or platforms. The SER must consider the loads that will be imposed by these systems in the design of the building.

Some systems include fall-restraint anchors that are intended to be cast into concrete knee walls or parapet walls; these walls must be designed to resist the loading from these anchors. Additional reinforcement is likely required locally at both the anchor-to-parapet connection and the parapet-to-slab connection.

In addition, very tall buildings will require building maintenance units, which are large cranes that sit on the roof of the building. These cranes have a significant mass and can impose very large gravity (and lateral) loads on the roof of the building; the depth of the roof slab and the size of the supporting columns may need to be increased.

The SER should encourage the project team to finalize the design of these systems during the design phase of the project, because it can be difficult and expensive to accommodate these elements in the structural design at the shop drawing stage of the project.

3.2.6.2 Elevators

In tall concrete buildings, the elevators are supported by machines that are located at the top of the elevator shaft and supported by the concrete slab that forms the lid of the elevator shaft. The loads exerted by the elevator machines can be significant, and the elevator manufacturer will have specific requirements for openings and flatness of this slab. The detailed design of this slab requires coordination with the elevator manufacturer, who should provide specific information in the form of shop drawings. The SER should anticipate that this slab might need to be at least 300 to 400 mm thick.

Horizontal displacement of a tall concrete building may cause distortion of the elevator shaft, which will reduce the vertical space available over the full height of the building for the installation of the elevator. The horizontal displacements may be permanent when they are due to GILD on the building, or may be transient when they are due to wind loading. It may be necessary to allow for a wider elevator shaft in order to accommodate the distortion.

For more information on the roles and responsibilities of professionals providing services related to elevating devices, refer to the *Professional Practice Guidelines – Professional Responsibilities for the Design and Installation of Elevating Devices in New Buildings* (Engineers and Geoscientists BC 2020b).

3.2.6.3 Temperature Effects

All materials, including construction materials, expand and contract with changes in temperature. The design of structural members that are not enclosed within the building envelope must allow for the expected thermal movement, or for stresses, in the case of restrained members.

Secondary structural elements, such as steel framing around mechanical units, are common and typically

outside of the conditioned space, which subjects them to more drastic temperature changes.

It is recommended to use slip connections from the steel to concrete to allow thermal movements to occur without inducing large stresses at the connection points.

3.2.6.4 Mass Concrete

Large footings or thick transfer slabs may require that special measures be taken to deal with the generation of heat from hydration of cement and the resulting volume changes.

The maximum temperature of the concrete and differential temperature strains must be limited to avoid cracking of the cooler surface concrete.

3.2.6.5 Cladding Support

The SER should specify assumptions with regard to gravity and lateral support of cladding systems on the structural drawings, to ensure finishes have adequate allowances for structural movements, and loading assumptions are clear for the Supporting Registered Professional (SRP), if retained.

Vertical members that span between floors may be required to support cladding systems where traditional mullions are not sufficient; for example, in buildings with large floor-to-floor heights. These vertical members are typically hollow structural sections; the SER should identify the size and location of these secondary structural elements during the design stage.

The SER should review and coordinate the vertical deflection allowance for the cladding system, to ensure that the cladding system will not be damaged by differential movement between two consecutive floor slabs.

Excessive seismic drifts need to be communicated to the Architect for consideration during the design of the cladding system. Some cladding systems, especially those with large panels, cannot tolerate large seismic drift demand (i.e., greater than 0.02).

3.3 DESIGN FOR LATERAL WIND FORCES

3.3.1 INTRODUCTION AND SECTION OVERVIEW

The effects of wind forces on tall concrete buildings must be considered during design, as wind often governs the required stiffness and strength of the LFRS.

This section provides considerations for the design of tall concrete buildings for lateral wind forces, including methods for calculating wind loads, and is broken down into the following sections:

- Section 3.3.2 Procedures for Calculating Wind Forces
- Section 3.3.3 SLS and ULS Criteria
- Section 3.3.4 Modelling Considerations
- Section 3.3.5 Strength Design of Lateral Force Resisting System
- Section 3.3.6 Supplementary Damping Systems

3.3.2 PROCEDURES FOR CALCULATING WIND FORCES

There are three procedures for calculating wind loads, and the Code prescribes when these procedures can be used:

- Static
- Dynamic
- Wind tunnel

Static Procedure: The static procedure is not appropriate for the design of tall concrete buildings because it does not consider the characteristics of the building. The wind loading design values predicted by the static method can be very unconservative when the dynamic characteristics of the structure are similar to those of the wind excitation. The original Tacoma Narrows Bridge (Washington state, United States, 1940) collapsed when the aeroelastic flutter caused by the wind matched the natural frequency of the structure. For this reason, wind loads on slender structures should be determined through model testing.

Dynamic Procedure: The dynamic procedure is recommended to be used to calculate wind lateral forces for tall concrete buildings, and is Code-mandated (Article 4.1.7.2. of the Code) where any of the following apply:

- the lowest natural frequency is between 1.0 and 0.25 Hz;
- the height is greater than 60 m; or
- the height of the building is greater than 4 times the effective width.

The "effective width" definition in the Code is based on empirical observations of when dynamic effects governed the wind response of buildings. However, these observations were mainly from rigid frame buildings where the LFRS was the entire width of the building. As such, the correlation of height-to-width ratio is not as applicable for shear wall buildings that have a lower frequency. For core wall buildings, the aspect ratio of the core walls (i.e., height of building to length of core walls in each direction) is generally used as an indicator of the building's slenderness.

Wind Tunnel Procedure: If the building's lowest natural frequency is less than 0.25 Hz, or if the height is more than 6 times the minimum effective width, the wind tunnel procedure (i.e., experimental procedure) is required.

In this case, the SER should inform the client of the requirement to engage a wind consultant to perform a wind study. This report should be prepared and authenticated by an Engineering Professional. The wind study should consider the adverse effect of proposed future developments, in addition to the current geographic and built environment specific to the building site.

For multiple towers linked by a podium, the concurrent effect of wind needs to be considered in order to capture the effect of building motions in the opposite direction. See subsection Podium Structures in Section 3.4.3.2 Seismic Demands for more information.

3.3.3 SLS AND ULS CRITERIA

The wind design of tall concrete buildings involves consideration of both the ULS criteria for adequate factored capacity to resist factored wind forces, and the SLS criteria for acceptable wind deflections and building movements.

For tall slender buildings, satisfying the maximum translation and rotational accelerations under wind excitation will generally govern the minimum stiffness requirements for the LFRS. Typically, in these buildings, the entire Primary Structural System can be considered in the calculation of wind deflections and vibration criteria. One of the SLS criteria is a limit of H/500 lateral deflection, as per Article 4.1.3.5. of the Code. Calculations of deflections must account for P-Delta effects.

The SLS criteria for occupant comfort under wind-induced accelerations can be one of the most challenging to satisfy. The perception of motion and the magnitude that is deemed acceptable is specific to individual occupants—design criteria will not satisfy all occupants all the time. The standard ISO 10137, Bases for Design of Structures – Serviceability of Buildings and Walkways Against Vibrations provides criteria for occupant comfort under wind vibration that specify the maximum accelerations during windstorm events with a 1-year return period. For the 1 in 10-year event, the Structural Commentaries to the *NBC* provides the acceptance range of wind accelerations between 1.5% to 2.5% of gravity, with the lower end of this range generally applied to residential occupancies and the higher end to business occupancies. It is recommended that the acceleration criteria fall within recommended limits for all of the 1-year, 5-year, and 10-year return period events. Since these criteria are related to the occupancy of the building, these limits are typically examined at the highest occupied floor in the building, and do not apply to maintenance spaces or unoccupied rooftops.

The wind consultant referred to in Section 3.3.2 Procedures for Calculating Wind Forces should prepare a report that provides detailed wind loads for the building (and cladding), and summarizes the expected wind vibration response (acceleration) and criteria for anticipated wind events with a 1-year, 5-year, and 10-year reoccurrence interval.

When a building does not meet the required SLS maximum acceleration criteria for occupant comfort, the dynamic response of the building can be altered by:

- reconfiguring the LFRS;
- generally adding stiffness;
- changing the building façade roughness (e.g., corner cut outs, balconies); or
- adding supplementary damping.

For very tall buildings, adding an open floor also can reduce vortex-shedding effects. Supplementary damping is discussed in Section 3.3.6 Supplementary Damping Systems.

3.3.4 MODELLING CONSIDERATIONS

Since the wind loading for tall concrete buildings is highly dependent on the dynamic response of the building, appropriate stiffness properties must be used for the analysis. As the building frequency decreases, the susceptibility to dynamic wind effects, such as vortex shedding, increases. Consequently, it is more conservative to use a lower frequency for the wind design and to avoid overestimating the stiffness of the building. The effect of foundation movement on building frequency should be considered and accounted for in computer modelling. Note that this effect is less pronounced in buildings with multiple below-grade levels.

The effective stiffnesses provided in CSA 23.3-14, under the commentary to Chapter 9, Structural Analysis and Computation of Deflections should be used as the initial values. However, the SER needs to confirm the actual effective stiffness based on the demands on the system, and whether the concrete will crack under the wind loading.

Typically, a tall concrete building is analyzed with two different computer models. The first model, for SLS response, includes both the LFRS and the Gravity-Load Resisting Frame members. Appropriate effective stiffnesses for SLS loads should be assigned to all members. The second model, for ULS wind design, includes only the LFRS elements, with the effective stiffnesses reduced to account for the increased cracking due to the higher ULS wind loads.

If the Gravity-Load Resisting Frame is used to help resist the ULS wind loads, effective stiffness values must be used for each of the members that reflect what portion of the wind forces are resisted by the two different systems. Thus, trial and error may be needed to determine the appropriate effective stiffness values.

For concrete shear wall buildings, the critical damping ratio for determination of dynamic wind effects (accelerations) is commonly taken as 2% for ULS loading and 1.5% for SLS loading. However, researchers have indicated that actual damping is lower in very tall slender buildings; for such buildings, the SER should consider using a lower damping ratio. There are a number of scholarly articles which provide guidance on this subject; see Section 6.0 References.

3.3.5 STRENGTH DESIGN OF LATERAL FORCE RESISTING SYSTEM

In high seismic zones, it is typical for the LFRS to be designed to resist all the ULS wind loads, and for the Gravity-Load Resisting Frame to be designed to be sufficiently flexible to tolerate the lateral displacements due to earthquake ground motion. If the Gravity-Load Resisting Frame is used to help resist the ULS wind loads, all members of the Gravity-Load Resisting Frame must be designed to resist the additional wind forces, and special consideration needs to be given to ensure the Gravity-Load Resisting Frame can tolerate the seismic displacements; see Section 3.4.6 Design of Gravity-Load Resisting Frames for Seismic Deformation Demands.

As buildings become taller, the wind forces increasingly govern the strength requirements of the LFRS. This results in a seismic over-strength of the concrete wall elements (such as coupling beams and wall piers), which introduces additional challenges to meeting the seismic Capacity Design requirements for the wall shear strength, foundation strength, and diaphragm strength; see Section 3.4 Design for Earthquake Ground Motions.

As the same concrete wall elements are used to resist the forces due to both wind and seismic demands, the stricter requirements for seismic design control the detailing of the reinforcement. For example, the vertical reinforcement used to resist tension forces from wind and seismic demands cannot be offset bent, according to the seismic design requirements. In some instances, such as the envelope of coupling beam design forces, the wind design requirements are stricter than the seismic design requirements. It is permitted to redistribute the coupling beam force demands from seismic demands as inelastic response; but the SER should avoid or limit the amount of redistribution of coupling beam forces from wind.

Wind forces generate significant diaphragm forces at the levels where the LFRS has a significant discontinuity (e.g., due to additional shear walls below a particular level*)*. CSA A23.3-14, which is referenced by the Code, does not have any provisions for the design of diaphragms for non-seismic loads. However, useful information on the design of diaphragms can be found in Clause 21 of CSA A23.3-14 (for seismic loads), or the new Clause 19 of CSA A23.3-19.

3.3.6 SUPPLEMENTARY DAMPING SYSTEMS

Common supplementary damping systems (dampers) for tall concrete buildings include:

- tuned mass dampers;
- tuned liquid sloshing and tuned liquid column dampers;
- viscoelastic coupling headers; and
- viscoelastic links in outrigger configurations.

Typically, these systems are designed by a wind consultant or design-build contractor, and they are tuned to suit the in-situ properties of the constructed building. Dampers can typically add approximately 1% to 2% additional damping, on top of the inherent damping that is assumed for the vibration response of the building. Dampers that rely on a large mass (approximately 0.5% to 1% of the total building mass) at the top of the building require a large amount of floor space. The mass of dampers must be accounted for in both the gravity and lateral (seismic) designs.

Tuned mass dampers may be expensive, due to the capital cost of the dampers, the cost of their design and installation, and the space they occupy at the top of the building (i.e., unoccupiable space).

Tuned mass dampers and water dampers are designed to work within a certain frequency range, to address serviceability concerns and affect the resonant response of the building. However, they cannot be relied upon to reduce the ULS wind response resonance case of the building, since the tuned water systems rely on the water being in the tank, and it may not be possible to guarantee this with sufficient reliability for the ULS case as well as the SLS case.

Integrated damping systems such as viscoelastic dampers and viscous dampers generally require less floor space than tuned mass dampers. Viscous dampers require maintenance and inspection, which is key to the reliability of these systems. Conversely, viscoelastic dampers do not require maintenance. Since these dampers are integrated into the building structure, they also affect the earthquake response of the building (generally in a favorable way). If the seismic analysis of LFRS takes advantage of dampers for earthquake response, the Non-linear Dynamic Analysis procedures discussed in Section 3.4.8 Evaluation of Life Safety Performance Using Non-linear Dynamic Analysis should be followed.

3.4 DESIGN FOR EARTHQUAKE GROUND MOTIONS

3.4.1 INTRODUCTION AND SECTION OVERVIEW

This section provides considerations for the design of tall concrete buildings for earthquake (seismic) demands including forces and displacements and is broken down into the following sections:

- Section 3.4.2 Preliminary Design Considerations
- Section 3.4.3 Determining Seismic Demands Using Linear Dynamic Analysis
- Section 3.4.4 Design of Concrete Shear Wall Cores
- Section 3.4.5 Refined Analysis of the Structure Below Plastic Hinge Zone
- Section 3.4.6 Design of Gravity-Load Resisting Frames for Seismic Deformation Demands
- Section 3.4.7 Advanced Design Issues
- Section 3.4.8 Evaluation of Life Safety Performance Using Non-linear Dynamic Analysis

3.4.2 PRELIMINARY DESIGN CONSIDERATIONS

3.4.2.1 Irregularities

A structure with simple, regular geometry is easier to design than an irregular structure, and will likely perform better during an earthquake. This does not mean that the building architecture must be simple and regular, but that the structure itself, particularly the SFRS, should be as regular as possible. A very irregular structure may have a higher probability of collapse during a large earthquake and is more likely to be damaged during a smaller earthquake.

The Code identifies a number of common irregularities, and has specific requirements for buildings with these irregularities; however, it should be noted that these additional requirements do not ensure that a highly irregular building will perform as well as a regular building. The additional requirements are intended to address life safety performance, not the level of damage that will occur.

The list of irregularities (Article 4.1.6.8. of the Code) is constantly being updated. For example, GILD is a new type of irregularity introduced in the current edition of Code, and sloped gravity-load columns is a new type of irregularity that will be included in the next edition of the Code, consistent with the *NBC 2020*. While the focus is often on irregularities in the SFRS, it is important to note that irregularities also exist in the Gravity-Load Resisting Frame, such as sloped gravity-load columns, which must also be accounted for in the design.

Another benefit of a structure with regular geometry is that it will have well-defined load paths for all forces, as well as a clear separation between the structural elements and structural actions that dissipate energy through Inelastic Deformation, and the structural elements and structural actions that must be capacity protected.

3.4.2.2 Geometry of Core

In a core wall building, the geometry of the core is crucial. The preferred location of the core from a structural perspective is at the centre of the building (strictly speaking, at the centre of mass). This is normally also the preferred location from a functional perspective, and thus most buildings have a central core.

In some cases, such as when the floor plate is very long or has multiple wings, the building may have several cores; in other cases, a single core may be located closer to one end of a building. In the latter case, additional lateral force resisting elements, such as shear walls, may be required to reduce the eccentricity of the SFRS. The location and geometry of the core and any additional shear walls should be coordinated with the Architect early in the project.

The core controls lateral drifts of the building due to wind forces and earthquake ground motions, and it resists lateral forces caused by wind, as well as seismic forces generated by earthquake ground motions. Thus, the core must have adequate lateral stiffness and lateral strength.

The core usually encompasses the elevators, the elevator lobby, and the stairways. One of the main factors that determines the size of the core, and hence the stiffness and strength of the core, is the number of elevators. Taller buildings typically have more elevators and, as a result, larger cores. Core walls have been used in British Columbia (BC) for more than 40 years, and generally, the functional requirements have resulted in appropriately sized cores. However, recent advances in elevator technology (i.e., increased speed and computer controls for more efficient demand management) is resulting in tall concrete buildings being constructed with fewer elevators. This results in more slender cores, which increases the challenge of designing buildings that will perform well. As a core becomes more slender, it becomes increasingly difficult to satisfy serviceability requirements for wind vibration, and to have sufficiently low seismic drift demands on the Gravity-Load Resisting Frame and on the nonstructural components in the building.

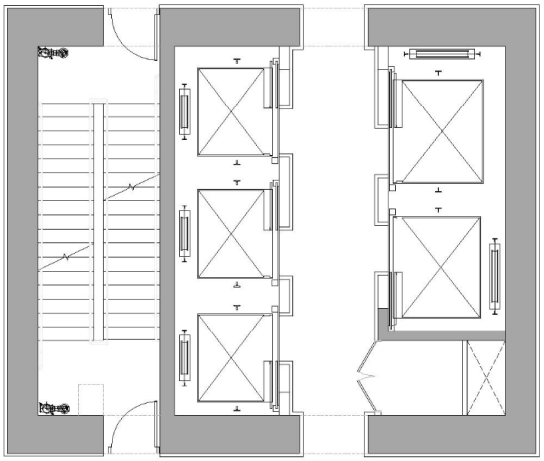
The preferred configuration of the core is a rectangular shape. This results in an orthogonal arrangement that avoids directional effects and simplifies the analysis of the seismic demands. For examples of core configurations, see Figure 1: Example cores from tall concrete buildings in British Columbia below.

A closed shape around the perimeter of the elevator and stair shafts results in good torsional rigidity. It is recommended that geometry of the core be such that the first two modes of the building are the lateral modes in the two perpendicular directions, and the third mode is the torsional mode. Additionally, it is recommended that there be good separation between the lateral modes and the torsional mode (i.e., a minimum of 0.5 s separation).

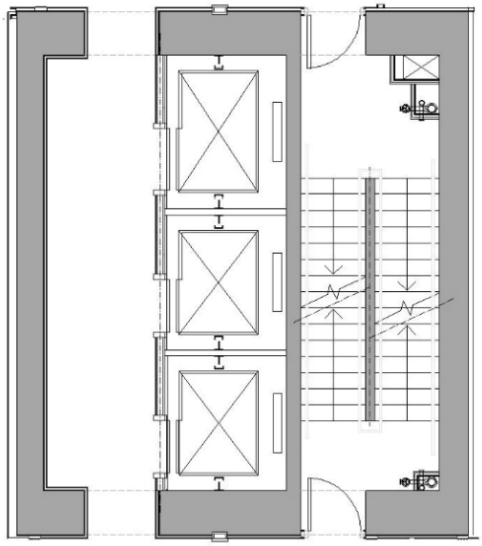
The GILD from dead load bending moments applied to the core must be considered when determining the arrangement of the core and the Gravity-Load Resisting Frame. When the core is located on one side of the floor plate so that dead load bending moments from the slab are applied on one side of the core only, the GILD can become very significant. Reducing the span of the slab will reduce the GILD. Similarly, columns that are placed at a similar distance on either side of a more centrally located core will reduce the GILD. When that is not possible, cantilever beams attached to the more lightly loaded side of the core can help reduce the unbalanced dead load moment. Finally, the GILD caused by column offsets may be reduced by providing counter-balancing offsets on the opposite side of the building.

A key characteristic of a regular core is uniform size and location of openings over the height of the building, particularly in the upper floor levels. Having solid walls where openings are stacked above in the upper floors can be tolerated below the plastic hinge; but solid walls are very undesirable in the upper floors as they will inhibit the coupling beam deformations needed for the good energy dissipation characteristics of a coupled wall system. The area of a missing opening (often referred to as a "panel zone") will have high shear demands that need to be carefully considered. To avoid this, the SER should consider maintaining openings in the concrete structure of upper floors and specifying that masonry block wall infill (with a sufficient gap at the boundaries) or drywall partition walls be used to fill the openings instead of introducing a solid wall.

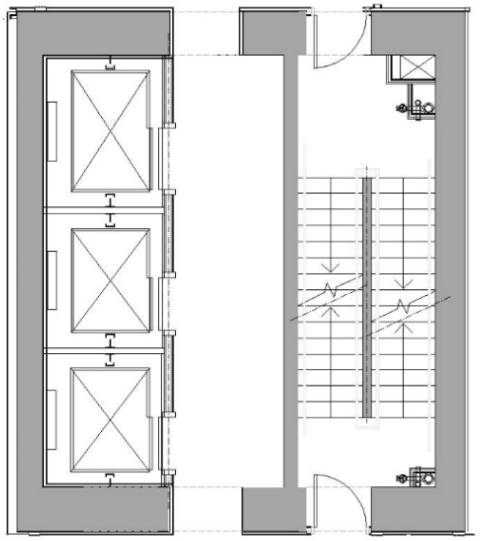
Finally, another key characteristic of an ideal core is that the walls have sufficient length to develop the diagonal reinforcement beyond the door openings. In a coupled wall system, the main energy-dissipating mechanism is the yielding of the diagonal reinforcement in the coupling beams. Thus, adequate anchorage of this reinforcement into the wall is essential. Note that CSA A23.3-19 has increased requirements whenever the full straight embedment length for the diagonal reinforcement is not provided; the SER should consider applying these additional detailing requirements proactively. Some of the example cores shown in Figure 1 include smaller-than-ideal wall piers at the ends of the coupling beams that would require special consideration in the design.



(A) Five-elevator core



(B) Three-elevator core



(C) Alternate three-elevator core

Figure 1: Example cores from tall concrete buildings in British Columbia

3.4.2.3 Gravity-Load Resisting Frame

While the SFRS is designed to resist all seismic forces and to control the building drifts, both the SFRS and the Gravity-Load Resisting Frame will experience the same lateral displacements of the building, as they are connected together by the floor slabs. Thus, the arrangement of the Gravity-Load Resisting Frame is a key part of the seismic design of a building.

A stiff SFRS (core) that limits the building drifts permits more architectural freedom in the arrangement of the Gravity-Load Resisting Frame; conversely, an SFRS that provides less drift control permits less architectural freedom in the Gravity-Load Resisting Frame. The deformation of the building induces demands on the Gravity-Load Resisting Frame in a number of different ways.

The column drift ratio (lateral displacement of column over a storey height) that results from the building drift ratio depends on the relative bending stiffness of the floor system and the columns. When the floor system is flexible (e.g., long span flat-plate slabs), very little bending is induced into the column. In that case, the slab is expected to crack around the column, thereby decreasing the punching shear resistance of the slab. Cracking of the slab starts reducing the punching shear resistance of a slab at a building interstorey drift ratio of 0.005 (0.5%); however, because of the different load factors for gravity design and seismic design, punching shear requirements tend to become significant when the building interstorey drift ratios exceed about 1.0%, as discussed in Section 3.4.6.5 Slab-Column Connections.

When the floor system has a high bending stiffness (e.g., when there is a thick transfer slab), the building drifts will generate large column bending, and since the columns are subjected to large axial compression, these members are usually not able to tolerate the induced bending demands. Any beams or curbs that frame into the gravity-load columns will further reduce the flexibility of these members because of their reduced length (i.e., the short-column effect). Stiff horizontal floor members may also generate significant axial force into the supporting columns, and this could cause failure of the column if not adequately accounted for.

As the building drifts are larger above the plastic hinge region, the influence of transfer slabs and transfer girders framing into the core is more significant. A large transfer element framing into the core may also influence where the plastic hinge will form in the building.

Locating the gravity-load columns as far as possible from the core increases the flexibility of the floor system. This reduces the bending induced into the columns and reduces the stiffness of the Gravity-Load Resisting Frame, and hence reduces the unintentional outrigger effect. Regardless of the column layout, when the slab has prestressing, there will be less flexural cracking of the slab and hence larger demands on the column.

See Section 3.4.6 Design of Gravity-Load Resisting Frames for Seismic Deformation Demands for further discussion of these topics. Example 4: Column In Close Proximity to Core in Appendix B: Non-linear Dynamic Analysis Examples presents the results of Non-linear Dynamic Analysis of a building with a gravity-load column in close proximity to the core.

Gravity-load bearing walls that are not intended to resist any seismic forces can still generate an irregularity if they have significant lateral stiffness. Similarly, non-structural walls, such as concrete exterior façade walls, may cause concentrations of deformations in the SFRS, and the façade walls may fail in a brittle manner if they do not have joints that can tolerate the required movements.

3.4.2.4 Concrete Strength

A number of design requirements motivate the use of higher concrete compression strength. The thickness of core walls is normally controlled by the maximum shear resistance of the wall ($V_{r,max}$), which is a function of the concrete compression strength. Higher concrete compression strength increases the (calculated) ductility of walls by reducing the compression strain depth; reduces the interstorey drift demands by

increasing the effective flexural rigidity ($E_c I_e$) of the walls; and reduces the required embedment length on the diagonal reinforcement in coupling beams.

CSA A23.3 limits the maximum compression strength of concrete used in an SFRS to 80 MPa; however, for a number of reasons, concrete strengths higher than 60 MPa tend not to be used very often for core walls in BC. Concrete with a strength higher than 60 MPa has a significant price premium. Another consideration is that the maximum concrete compression strength that can be used to determine the concrete contribution to shear resistance of a wall (V_c) is limited to 64 MPa, even though the concrete contribution to shear resistance is typically not large in the plastic hinge region.

The Code equation for modulus of elasticity of concrete (E_c), which affects the flexural rigidity of shear walls, has been found to be generally conservative in BC due to the good quality aggregate found here.

3.4.2.5 Reinforcement Strength

The CSA A23.3 provisions for concrete structures, with the exception of the confinement reinforcement requirements, were developed for Grade 400 reinforcement (yield strength of 400 MPa). The additional elastic strains that result from using higher-grade reinforcement will generally reduce the ductility of concrete structures and may reduce the shear strength. As such, CSA A23.3 requires that the design, detailing, and ductility requirements for structures designed using a reinforcement grade higher than 400 MPa account for the increased strain demands. The complexity of accounting for the increased strain demands effectively prohibits the use of higher yield strength reinforcement at this time.

Grade 500 MPa reinforcement can be used for confinement ties of vertical column reinforcement, and can be used in capacity-protected elements such as core footings and raft foundations that are not shear-critical. One advantage of higher-grade reinforcement is reduced congestion.

Future editions of CSA A23.3 are expected to contain provisions for the use of higher-grade reinforcement.

3.4.3 DETERMINING SEISMIC DEMANDS USING LINEAR DYNAMIC ANALYSIS

3.4.3.1 Modelling Requirements

Mass and Stiffness

The model of the building must represent the actual mass and stiffness of the real structure as closely as possible. The mass must be correct in magnitude and distribution in all three dimensions. For example, the mass of a water tank on the roof of the building and the mass of landscaping on a podium slab must be modelled at the correct elevation and location on the floor plan.

Determining the effective stiffness of a concrete building during an earthquake is complex, because the stiffness of the building changes as cracks form due to the imposed deformations. Thus, the Code requires the use of both an upper-bound and a lower-bound estimate of the stiffness. The minimum design force level is determined using an upper-bound estimate (i.e., the stiffness of the building before significant damage). This is the empirical fundamental lateral period prescribed by the Code. Conversely, the displacement demands must be determined using an estimate of stiffness that accounts for the expected level of damage. CSA A23.3 defines the effective stiffness of concrete shear walls and coupled walls as a function of the elastic bending moment demand to the strength of the wall. The lower the strength, the lower the effective stiffness, because a lower strength wall will sustain more damage (i.e., more cracking) during the earthquake.

As the flexural strength of the concrete walls is not known when the analysis is started, a lower-bound estimate of strength must be used to make a lower-bound estimate of stiffness. A refined (increased) stiffness can be used after the design is completed, and the strength is known, to make refined (reduced) estimates of building displacements if needed.

Subterranean Levels

When the seismic base is at grade and the site is not sloping, the mass of the subterranean structure (including grade level) may be ignored when doing the Linear Dynamic Analysis of the building, in order to get a correct estimate of the base shear at grade level.

A complete model of the subterranean structure needs to be included in the building model, in order to account for the flexibility of the base of the building. Upper-bound estimates of stiffness (e.g., uncracked stiffness) of the subterranean structure can be used, and the flexibility of the soil or rock below the foundation can be ignored, as CSA A23.3 requires that a refined model of the subterranean levels with a range of stiffness assumptions be used to determine the forces in the elements below grade (see Section 3.4.5 Refined Analysis of Structure Below Plastic Hinge Zone). This refined analysis will also be used to determine the additional drifts in the tower walls due to cracking of the below-grade structure and movement of the foundation resulting from flexibility of soil or rock. The additional deformations determined in the refined analysis of the structure below the plastic hinge zone need to be added to the results of the building dynamic analysis.

The subterranean floor diaphragms that connect the lateral force resisting elements (core walls and perimeter walls) must be modelled as semi-rigid in both analyses.

On large projects, a significant grade difference can occur from one side of the site to the other; this can impact the modelling assumptions and the soil loading that must be considered.

Diaphragms

The concrete diaphragms can be modelled as rigid for many levels of a tall concrete building, which simplifies the analysis of the building.

The diaphragm must be modelled as semi-rigid in the following situations:

- At any level where there is a discontinuity in the wall stiffness. The floor above and below the discontinuity may also need to be modelled as semi-rigid.
- At any level where a concrete wall terminates.
- At the subterranean levels where the diaphragms connect the core walls and the perimeter foundation walls.

Example 3: Discontinuous Shear Wall in Appendix B: Non-linear Dynamic Analysis Examples presents the results from Non-linear Dynamic Analysis of a building with a discontinuous shear wall.

Gravity-Load Resisting Frame

The Code requires that the SFRS be designed to resist 100% of the lateral earthquake loads. Thus, the analysis of the SFRS must be done with a model that eliminates any earthquake loads being resisted by the Gravity-Load Resisting Frame. A simple way to achieve this in the model is to provide hinges at the top and bottom of all gravity-load columns and walls, and reduce the out-of-plane bending stiffness of the slabs.

Note that unless the Gravity-Load Resisting Frame is very flexible, additional analyses with different models for the Gravity-Load Resisting Frame must be done to investigate any potential detrimental influence that the Gravity-Load Resisting Frame might have on the seismic response of the building. The Code requires that if the stiffness of the Gravity-Load Resisting Frame, using a best-estimate model of the frame, decreases the fundamental lateral period of the building by more than 15%, the period of the building used to determine the minimum design forces must account for the Gravity-Load Resisting Frame. That is, the building must be designed for higher force levels.

In addition, if the Gravity-Load Resisting Frame introduces or increases the irregularity of the building, or has any other adverse effect on the SFRS, the detrimental influence of the Gravity-Load Resisting Frame must be accounted for in the design of the SFRS.

Finally, the Gravity-Load Resisting Frame must be investigated and shown to behave elastically or to have sufficient non-linear capacity to support the gravity loads while undergoing earthquake-induced deformations. The separate analysis that needs to be done to satisfy this last requirement is discussed in Section 3.4.6 Design of Gravity-Load Resisting Frames for Seismic Deformation Demands.

P-Delta

For tall concrete buildings, second-order (P-Delta) bending moments must be correctly accounted for in the analysis of the building. These additional bending moments cause additional lateral displacements (drift) of the building.

Computer programs for the Linear Dynamic Analysis of buildings have different options for how the P-Delta is determined. Generally, the iterative procedure in which the load is computed from a specified combination, known as the P-Delta combination, must be used, because the Code specifies that the companion loads (0.5 for live loads and 0.25 for snow loads) must be considered. The iterative procedure considers P-Delta on an element-by-element basis, and local buckling is captured more effectively; however, there may be more difficulty with convergence.

The alternative mass-based, non-iterative procedure in which the load is calculated from the mass at each level is an approximate procedure that may underestimate the P-Delta effect due to the difference between the mass and the total gravity loads.

3.4.3.2 Seismic Demands

Site Class

A key factor that influences the seismic demands on a building is the site class. The average shear wave velocity in the top 30 m of soil or rock (V_{s30}) is a parameter that is used to determine the site class.

The Code does not clearly define the "top" from where the 30 m is to be measured, and different interpretations have been used in the past. Based on recent consensus of good professional practice, the 30 m should be measured from the ground surface.

Note that the LATBSDC guidelines indicate that when the total weight of the building including the subterranean floors exceeds the weight of soil removed for construction of the subterranean levels, the "top" of the 30 m may be taken half-way between the foundation level and the ground level (LATBSDC 2020). This approach contradicts the recommended practice mentioned above and should be used with caution.

When a site has strong soil/rock impedance contrast (i.e., shallow bedrock), the measured V_{s30} may give inaccurate measures of the site amplification or de-amplification of ground motions. In that case, a site-specific response spectrum should be considered; see subsection Site-Specific Response Analysis in Section 3.4.8.4 Seismic Hazard.

Scaling Linear Dynamic Analysis Results

The minimum seismic design forces prescribed by the Code depend on an empirical period that is a function of the height above the base, where the base of the structure is the level at which the horizontal earthquake motions are considered to be imparted to the structure. Depending on the size and stiffness of the subterranean structure, the earthquake motions will be imparted to the structure somewhere between the foundation level and grade level. Measuring the height of the building from grade level results in a safe estimate of the minimum seismic design forces.

If the structural model of the building, accounting for cracking of concrete, indicates a longer fundamental lateral period of the building, as it normally does, the fundamental lateral period used to determine the minimum lateral earthquake forces can be increased by up to a factor of 2.0 times the empirical value.

The maximum fundamental lateral period that can be used to determine the minimum seismic design forces is 4.0 s (for concrete shear wall buildings). Assuming the factor of 2.0 described above is applicable, this limit is reached when the height of a building exceeds 137 m. Taller buildings will have reduced ductility demands resulting from the increased minimum lateral earthquake forces.

Article 4.1.8.7. of the Code defines a scaling factor for displacements whenever the fundamental lateral period of the building, as determined from the structural model that includes the influence of damage due to earthquake motions, is greater than 4.0 s. The scaling factor for displacements, determined from the minimum lateral earthquake force for a fundamental lateral period of 4.0 s, will be different from the scaling factor for forces if the height of the building is less than 137 m (and the factor of 2.0 described above is applicable).

The Code states that intermediate values of design spectral acceleration (between explicitly defined points) can be determined by linear interpolation. As the ratio of spectral displacement to spectral acceleration is proportional to T^2 (where T is the fundamental lateral period of the building), the linear variation of acceleration between defined points results in parabolic variations of displacement. The large period range between defined points at 2 s, 5 s, and 10 s results in a very distorted displacement spectrum. Thus, either the design spectral displacement values should be determined using linear interpolation between the defined points at T = 0.2 s, 0.5 s, 1 s, 2 s, 5 s, and 10 s (a piece-wise linear displacement spectrum used to back-calculate the acceleration spectrum), or log-log interpolation should be used for the acceleration spectrum, as suggested in *NBC 2020*.

The results of the Linear Dynamic Analysis must be scaled to 100% of the minimum lateral earthquake force if the building has any of the nine Code-defined irregularities. Vertical stiffness irregularity commonly occurs when concrete walls are terminated part way up the height of a building. Mass irregularity may occur, for example, due to the mass of landscaping on a podium floor level. If the building does not have any of the Code-defined irregularities, the results of the Linear Dynamic Analysis may be scaled to 80% of the minimum lateral earthquake force.

The minimum lateral earthquake force is based on a simple static model of the structure that is constrained to move only in the direction of the earthquake motion. Therefore, the dynamic analysis model that is used to determine the scaling factor must be similarly constrained, preventing translation of the building in the other direction, as well as any rotation of the building. The scale factor is determined by dividing the lateral earthquake forces obtained from the simple static model (either 100% or 80%, as described above) to the constrained dynamic model of the building. Once the scale factor has been obtained, it is applied to the forces and/or displacements obtained from a dynamic analysis of the unconstrained model. Note that the scaling factor obtained in this way will be smaller for more torsionally eccentric buildings.

When the value of the base shear determined from dynamic analysis is larger than the minimum base shear calculated using the static procedure, the base shear from the dynamic analysis must be used. This will occur when the dynamic model is stiffer than the static model (resulting in a smaller value of the fundamental lateral period) or where higher modes dominate the dynamic response. An example of the latter condition is a flexible, long-period tower on top of a large, heavy, short-period podium.

Torsion

When determining whether a building has a torsional sensitivity irregularity, any Gravity-Load Resisting Frame members (such as walls) that influence the torsional deformations of the building must be considered in the analysis. Torsional sensitivity is common in buildings with long floor plates.

Torsional sensitivity increases the design forces in two ways:

1. as an identified irregularity, where the base shear must be scaled to 100% of the minimum lateral earthquake force; and
2. where larger accidental torsion demands must be included in design.

Two different procedures are used to account for the effects of accidental torsional moments acting concurrently with the lateral earthquake forces:

- In the static procedure, larger demands result from the static effect of torsional moments due to the lateral force at each level being shifted ±10% of the plan dimension of the building perpendicular to the direction of seismic loading being considered.
- When a building is not torsionally sensitive, the dynamic procedure with a three-dimensional dynamic analysis of the building can be used, with the centres of mass shifted by a distance equal to 5% of the plan dimension of the building perpendicular to the direction of seismic loading being considered.

It is good practice to use the static procedure for determining the demands from accidental torsion whenever the first mode of the building is a torsional mode, irrespective of whether the building has a torsional sensitivity irregularity.

Podium Structures

Special consideration is needed when designing tall concrete buildings on top of podium structures that have significantly greater lateral stiffness and mass, particularly when multiple towers share a common podium structure. The challenges include:

- estimating the fundamental lateral period used to determine the minimum base shear;
- scaling the results of the dynamic analysis of the building to the minimum base shear;
- selecting the proper force reduction factors R_d R_o when lower ductility shear walls (e.g., squat walls) are part of the podium structure; and,
- determining the demands on the diaphragms that interconnect the tower walls and the podium walls.

When two towers are connected by a podium structure, the following approach can be used to scale the results from Linear Dynamic Analysis:

1. Use a constrained dynamic model of Tower 1 and the podium structure to determine the scale factor applied to forces in Tower 1 above the podium levels.
2. Use a constrained dynamic model of Tower 2 and the podium structure to determine the scale factor applied to the forces in Tower 2 above the podium levels.
3. Use a dynamic model that includes Tower 1, Tower 2, and the podium structure, and the larger of the two scaling factors above, to determine the design forces in the podium levels.

Example 1: Two Towers on a Common Podium in Appendix B: Non-linear Dynamic Analysis Examples presents the results from the Non-linear Dynamic Analysis for this type of structure.

3.4.4 DESIGN OF CONCRETE SHEAR WALL CORES

This section presents aspects of the design of concrete shear wall cores including:

- determination of the element design forces;
- design of the coupling beams, which must be done before the design of wall piers to resist axial load and bending moment;
- the factored, nominal, and probable flexural overstrength of walls;
- design of walls for shear;
- the wall ductility check; and
- a summary of detailing requirements, including transferring of horizontal wall forces across construction joints.

3.4.4.1 Element Design Forces

To determine the force demands on the elements of the SFRS, the model of the building is used where only the SFRS resists the earthquake loads and effects (with hinges at the top and bottom of all gravity-load columns and walls, and reduced out-of-plane bending stiffness of the slabs). The results include bending moments and shear forces applied to all elements (wall piers and coupling beams), and axial loads applied to wall piers in the coupled wall direction.

In addition, axial loads applied to the wall piers due to gravity loads must be determined. The axial compression forces due to gravity loads reduce the net uplift in the wall piers, and therefore reduce the amount of vertical reinforcement required in the wall piers to resist the earthquake loads. This suggests that a lower-bound estimate of axial compression would be safe.

However, axial compression forces due to gravity loads also reduce the inelastic rotational capacity of the wall piers, and therefore decrease the ductility of the wall system. Thus, an estimate based on the best available information (rather than a lower-bound or upper-bound estimate) should be made for the gravity loads on the wall piers. See Section 3.2.2.3 Estimating Load Distribution to Columns and Bearing Walls for a discussion of the calculation of gravity loads on columns and walls using three-dimensional Linear Analysis of the floor system, versus using a simple floor-by-floor tributary area approach.

According to the Code, the factored loads to be considered with the earthquake loads ($1.0E$) is 100% of dead loads ($1.0D$) and, when detrimental, is 50% of live loads ($0.5L$) and 25% of snow load ($0.25S$). The load combination $1.0E + 1.0D$ is usually critical for the design of the vertical reinforcement, while $1.0E + 1.0D + 0.5L + 0.25S$ is usually critical for the ductility check.

Unlike for wind design, the Code does not require that two different dead load factors be considered for seismic design. When a significant allowance for dead load is required for components that may or may not be installed or constant throughout the life of the building—for example, landscaping, partition walls, or floor levelling—it may be appropriate to use two estimates of the dead load for seismic design, to determine the most conservative load combination.

The elements of the SFRS must also be designed to resist the lateral loads due to wind; for tall concrete buildings, this often controls the required strength of the core, particularly the coupling beams. The factored dead loads to be considered with factored wind loads ($1.4W$) is either 125% or 90% of dead load ($1.25D$ or $0.9D$), whichever is more critical. In addition, when additional gravity loads are detrimental, 50% of live loads ($0.5L$) and 50% of snow load ($0.5S$) must be included (Article 4.1.3.2. of the Code). Note that when a building has a GILD, the increased gravity loads due to $1.25D + 0.5L + 0.5S$ will often cause larger lateral force demands on the walls than those required to be considered for wind design.

3.4.4.2 Design of Coupling Beams

The design of the coupling beams must be completed before the wall piers can be designed, because the wall piers must meet the CSA A23.3 capacity design requirement of not yielding before significant yielding has occurred in the coupling beams.

Efforts should be made to balance the opposing requirements of providing minimum overstrength, while providing a simple variation of reinforcement over the height in order to simplify construction.

The coupling beam capacity must be at least equal to the demands from wind loading (1.4*W*) for that storey; that is, yielding of the coupling beams and redistribution of the coupling beam forces to adjacent levels is not permitted. Conversely, for the earthquake loading case (1.0*E*), a limited amount of redistribution of elastic coupling beam forces is considered acceptable. Consistent with the commentary to CSA A23.3, it is good practice to limit the maximum redistribution of coupling beam forces from earthquake to 20%.

Diagonal reinforcement is most commonly used in the design of coupling beams to resist the entire factored shear force and factored bending moments. However, when the coupling beam satisfies certain dimensional limitations and its shear stress level is sufficiently low, it is possible to design ductile coupling beams using longitudinal and transverse reinforcement, similar to a ductile beam in a moment-resisting frame.

CSA A23.3 specifies a number of dimensional limitations for diagonally reinforced coupling beams. Such limitations include that the centroid of each group of diagonal reinforcing bars must be centred in the beam, the beam must be centred on the wall pier, and the beam width must be less than or equal to the wall thickness. The amount of diagonal reinforcement and these requirements usually dictate the thickness of the wall piers in the coupled wall direction.

Anchorage of Diagonal Reinforcement

Editions prior to CSA A23.3-14 required that the wall piers at each end of the coupling beam must have sufficient length so the diagonal reinforcement can be anchored into the wall with a minimum straight embedment of 1.5 times the development length of the reinforcing bars.

In CSA A23.3-14, the requirement for a minimum straight embedment length was relaxed, and the standard now permits using a reduced straight embedment length plus a standard hook contained within "confinement reinforcement." Note that the term "confinement reinforcement" is a defined term in CSA A23.3 that requires significantly more transverse reinforcement than is usually provided in a concrete wall. In addition, headed and mechanically anchored reinforcing bars are permitted to supplement a reduced straight embedment length, provided data is available on the seismic performance of the head or mechanical anchorage in tension and compression, and this data is used to determine the anchorage requirements.

CSA A23.3-19, which will be adopted in the next edition of the Code, has new requirements related to the design of coupling beams; the SER is strongly encouraged to consult CSA A23.3-19 and consider adopting those new conservative provisions proactively. Of particular importance is the new requirement to explicitly design the beam-wall joint to transfer the probable shear force and probable bending moment from the coupling beam to the wall pier. The beam-wall joint will experience internal forces similar to a beam-column joint in a moment-resisting frame.

The zone where the diagonal reinforcement intersects with the wall pier is a critical zone, because the yielding portion of the diagonal reinforcement will penetrate into the wall in this region. CSA A23.3 requires that the anti-buckling hoops on the diagonal reinforcement extend into the wall at least the compression development length of the diagonal reinforcement, unless the diagonal reinforcement passes through buckling-prevention ties on vertical reinforcement. It is common practice to ensure the

zone of vertical reinforcement with buckling-prevention ties is large enough so that hoops are not required on the diagonal reinforcement in the wall.

Confinement Reinforcement in Coupling Beams

Research in the United States has resulted in the adoption of an alternative method for constructing diagonally reinforced coupling beams, as described in ACI 318, Building Code Requirements for Structural Concrete. In lieu of providing anti-buckling ties on the diagonal reinforcement, confinement reinforcement is provided around the perimeter of the entire coupling beam.

The intent of this alternative approach is to simplify construction by allowing the diagonal reinforcement to be "threaded into place" after the wall reinforcement is in position. Special attention is needed to ensure all untied diagonal reinforcing bars are placed within the required small tolerance on the angle of the diagonal reinforcing bars. A small change in angle of the diagonal reinforcement may have a significant influence on the resistance of the coupling beam.

CSA A23.3 has not adopted this alternative reinforcement arrangement for use in Canada; headers must be confined as described in CSA A23.3.

Openings in Coupling Beams

In high seismic areas such as the Lower Mainland of BC and Vancouver Island, it is good practice to restrict mechanical openings in the coupling beams. A common example of this is to limit the openings to two maximum 50 mm openings for small pipes such as sprinkler pipes, provided the openings do not interfere with the diagonal reinforcement in the header.

However, the SER must use professional judgment to determine the appropriate limitations for specific projects.

3.4.4.3 Design of Wall Piers for Axial Load and Bending Moment

Coupled Wall Piers

An important stipulation for the design of core walls is the capacity design requirement that wall piers must not yield in tension due to uplift from coupling beam shear forces before significant yielding has occurred in the coupling beams. Yielding of the coupling beams is the intended energy dissipation mechanism. CSA A23.3-19 provides the most comprehensive explanation of the requirements.

The axial forces in the wall piers of coupled wall systems are the result of the coupling beam shears. The Linear Dynamic Analysis of the wall system provides an envelope of the coupling beams shears, and an envelope of the axial forces in the wall piers associated with the coupling beam shears. The first mode generates an axial force in the wall pier that is equal to the sum of the coupling beam shear for that mode. Higher modes have coupling beam shear forces acting in opposite directions, which reduces the associated axial force in the wall pier.

CSA A23.3 requires that at each level, the axial force determined from the Linear Dynamic Analysis must be increased by the ratio of the "sum of the coupling beam nominal shear resistances above that level" to the "sum of factored shear forces in coupling beams above that level." The sum of the coupling beam shear strengths (or flexural strengths) is used to calculate the over-strength factor; this factor is used to increase the axial forces in the wall piers. The axial forces in the wall piers are output from the Linear Dynamic Analysis (which accounts for the higher mode effects). That is, the influence of higher modes in reducing the axial force in the wall piers is accounted for.

Cores act as a closed tube to resist torsion. The shear flow around the tube increases the shear force demands on the coupling beams. When determining the overstrength of the coupling beams, the shear force demands from accidental torsion should not be included; this decreases the magnitude of the

denominator ("sum of factored forces in coupling beams") and therefore increases the overstrength of the coupling beams.

Plastic Hinge Region

Once the axial forces applied to the wall piers from the coupling beam shear forces have been adjusted based on the nominal overstrength of the coupling beams, the reinforcement required in the wall piers can be confirmed at the critical section. The critical section of the wall piers is at the base of the plastic hinge region, where the vertical reinforcement will first yield. The vertical reinforcement is confirmed by comparing the axial force–bending moment interaction diagrams from plane sections analyses of the wall piers, with the various combinations of axial loads due to gravity loads, and axial loads and bending moments due to wind and earthquake.

CSA A23.3 states that the properties of the wall cross section that affect the bending resistance of the wall must be maintained over the height of the plastic hinge, thereby suggesting that no wall openings are permitted in the plastic hinge zone. However, the explanation in CSA A23.3-19 has been revised to suggest that this is not the case. The new explanation states that the quantity of vertical reinforcement required at the critical section must not be reduced over the height of the plastic hinge, and the bending resistance of the wall must be maintained without abrupt changes. Depending on the size and location of wall openings, additional vertical reinforcement may be provided to prevent a concentration of inelastic demands at wall openings. A large opening in the wall could significantly reduce the bending resistance over a portion of the plastic hinge height and cause a concentration of larger inelastic demands over that reduced height, which is not acceptable.

Above Plastic Hinge

The factored bending moment envelopes determined from a Linear Dynamic Analysis of the building must be adjusted to ensure that flexural yielding of the wall will not first occur above the plastic hinge region. Yielding must occur within the plastic hinge region, as that is where the special detailing is provided to ensure a ductile inelastic response.

The factored bending moments of individual wall piers at all elevations above the plastic hinge region must be increased by an overstrength ratio calculated by dividing the "factored bending moment resistance of the core" by the "factored bending moment demand on the core," both calculated at the top of the plastic hinge region. For a discussion on how to calculate the resistance of the entire core in the coupled wall direction, see Section 3.4.4.4 Factored, Nominal, and Probable Flexural Resistances of Core Walls.

The overstrength ratio at the top of the plastic hinge region includes the overstrength of the core at the base of the plastic hinge, the additional overstrength resulting from the constant vertical reinforcement over the height of the plastic hinge region, and the reduction in factored bending moment over the height of the plastic hinge region.

Note that the axial forces in the wall piers of coupled wall systems that result from the coupling beam shears are adjusted separately from the bending moment envelopes over the height. As mentioned above, at each level, the axial force determined from the Linear Dynamic Analysis is increased by the ratio of the "sum of the coupling beam nominal resistances above that level" to the "sum of factored forces in coupling beams above that level."

3.4.4.4 Factored, Nominal, and Probable Flexural Resistances of Core Walls

Three different types of flexural resistances need to be calculated for the reinforced concrete core. These are summarized in Table 1: Types of Flexural Resistance below.

Cantilever Walls

The flexural resistances of cantilever wall piers can be calculated either for individual wall piers (i.e., complete wall pier including "web" and "flange" portions), or they can be calculated for the complete core (i.e., all wall piers combined).

Coupled Walls

The factored, nominal, and probable flexural resistances of highly coupled core walls (i.e., with two or more openings) is a system property that cannot be determined by looking at individual wall piers, as can be done in the cantilever direction or for a lightly coupled wall (i.e., with one opening) system.

Figure 2: Freebody diagram to determine the flexural resistance of three coupled wall piers below summarizes the forces at the critical section (location of maximum bending moment) of a highly coupled core wall system with three wall piers, subjected to an overturning moment acting in the clockwise direction. The system needs to be analyzed separately for each of the three types of flexural resistance: factored, nominal, and probable.

The following is a description of the component forces acting on the wall piers:

- P_1, P_2, and P_3 are the axial loads in the three wall piers due to dead load (1.0D).
- V_{12} is equal to the axial force in wall pier 1 from Linear Dynamic Analysis times the ratio of "the sum of pier 1–pier 2 coupling beam (factored, nominal, or probable) shear resistances over full height" to 'the sum of factored shear forces in coupling beams without accidental torsion."
- T_1 is the tension force required in wall pier 1 for equilibrium of vertical forces on wall pier 1. T_1 may be limited by the (factored, nominal, or probable) strength of the vertical reinforcement in pier 1, and in that case, the value of V_{12} must be reduced accordingly. Otherwise, T_1 is defined by the capacity of the coupling beams, not by yielding of vertical reinforcement.
- V_{23}, at the compression end of the coupled wall, is equal to the axial force in wall pier 3 from Linear Dynamic Analysis times the ratio of "the sum of pier 2–pier 3 coupling beam (factored, nominal, or probable) shear resistances over full height" to "the sum of factored shear forces in coupling beams without accidental torsion."
- C_2 and C_3 are defined by the capacity of the coupling beams and are determined from equilibrium of vertical forces in the individual wall piers.
- M_1, M_2, and M_3 are determined by doing three separate sectional analyses (to determine the maximum) of the individual wall piers subjected to the net axial forces T_1, C_2, and C_3. As the three wall piers are interconnected by rigid floor slabs, they must have compatible curvatures. That is, M_1, M_2, and M_3 must be occurring at the same curvature value. The overturning resistance of the core in the coupled-wall direction is primarily due to the axial forces in the two exterior wall piers (T_1 and C_3). The flexural resistance of the individual wall piers (M_1, M_2, and M_3) generally provide a small contribution towards the flexural resistance of the entire core.

Table 1: Types of Flexural Resistance

TYPE OF FLEXURAL RESISTANCE	CONCRETE STRENGTH	REINFORCEMENT STRENGTH	HOW USED
Factored (M_r)	$0.65f_c'$	$0.85f_y$	See Note a
Nominal (M_n)	$1.0f_c'$	$1.0f_y$	See Note b
Probable (M_p)	$1.0f_c'$	$1.25f_y$	See Note c

NOTES:

[a] The factored flexural resistance must be greater than the factored flexural demand.

[b] The nominal flexural resistance is needed for:

- refined estimate of effective stiffness (if needed);
- evaluating ductility of cantilever shear walls (see Section 3.4.4.6 Ductility of Walls);
- refined analysis of subterranean or podium structure (see Section 3.4.5 Refined Analysis of Structure Below Plastic Hinge Zone); and
- shear force demands for moderately ductile walls.

[c] The probable flexural resistance of the core is needed for:

- shear force demands for ductile walls (see Section 3.4.4.5 Design of Walls for Shear); and
- refined analysis of subterranean or podium structure (see Section 3.4.5 Refined Analysis of Structure Below Plastic Hinge Zone).

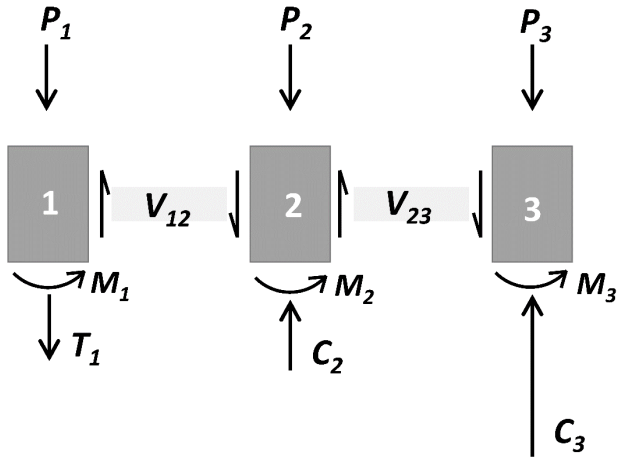


Figure 2: Freebody diagram to determine the flexural resistance of three coupled wall piers

3.4.4.5 Design of Walls for Shear

One of the most critical aspects of the seismic design of a concrete shear wall building is the shear design. This includes making a safe estimate of the shear force demands over the height of the building and calculating the shear resistance of the wall, accounting for the deformation demands in the plastic hinge region.

Shear Force Demands

CSA A23.3 requires that the factored shear force demand determined from a Linear Dynamic Analysis of the building must be increased to account for flexural overstrength of the wall, and further increased to account for inelastic effects of higher modes.

To account for flexural overstrength, the factored shear force envelope over the height of the building is increased by the ratio of "the (probable or nominal) bending moment capacity" to "the factored bending moment demand," both calculated at the base of the plastic hinge region. The probable flexural overstrength is used for ductile ($R_d \geq 3.5$) wall systems, while nominal flexural overstrength is used for moderately ductile ($R_d \leq 2.5$) wall systems.

To account for inelastic effects of higher modes, the factored shear force envelope over the height of the building must be further increased by the higher mode shear amplification factor, since the force modification factors ($R_d R_o$) used to determine the design bending moment (accounting for flexural ductility) are also used to determine the design shear force. Flexural yielding at the base of a wall limits the first mode shear forces but does not limit the higher mode shear forces. The higher mode shear amplification factor is a correction to the force modification factors for shear force to account for the additional shear forces (from higher modes) that are transmitted through the plastic hinge at the base.

The parameter that influences the higher mode amplification factor is the net force reduction factor ($R_d R_o / \gamma_{ws}$), where γ_{ws} is the wall overstrength factor for shear equal to the ratio of "nominal flexural resistance of the wall system" to "factored bending moment on the wall system." Any flexural overstrength of the wall reduces the amplification that is required to account for higher mode shear forces. When calculating γ_{ws}, the factored bending moment may be calculated using the design base shear prior to scaling to the minimum lateral earthquake force. That is, the over-strength due to scaling the base shear may be accounted for in the net force reduction factor.

CSA A23.3 limits the amplification factor for inelastic effects of higher modes to a maximum value of 1.5, even though Non-linear Response History Analysis indicates the amplification can be considerably larger. The reasons for this are:

- the maximum shear force occurs only once during an earthquake and lasts for a very short time;
- well-detailed concrete walls have shear ductility; and
- the maximum shear force generally does not occur when the base rotation is maximum, while the CSA A23.3 shear design procedures for concrete walls assumes that it does (Adebar 2018).

The amplification factor for higher mode shear forces does not apply to coupled and partially coupled walls.

Example 5: Inelastic Effects of Higher Mode Shears in Appendix B: Non-linear Dynamic Analysis Examples compares the results of Non-linear Dynamic Analysis with the shear force demands according to CSA A23.3.

Shear Resistance – Plastic Hinge Region

CSA A23.3 expresses the shear resistance of a reinforced concrete member as the sum of the steel contribution (V_s) and the concrete contribution (V_c), which is related to the shear that can be transmitted across the critical diagonal cracks. To avoid diagonal crushing of concrete, CSA A23.3 limits the shear strength to $V_{r,max}$. Shear design is often the critical aspect of the seismic design of tall concrete shear walls; as such, $V_{r,max}$ is usually the most critical aspect of shear design.

Figure 3 below shows the plastic hinge region of a concrete shear wall subjected to lateral load acting to the right. A simple strut-and-tie model describes how

the forces resisted by the horizontal reinforcement (i.e., the steel contribution, V_s) are assumed to flow in the plastic hinge region of the wall. Within the shaded triangular area (the "fan" region) above the critical flexural crack at the base, the diagonal compression struts concentrate the shear force in the flexural compression zone. The critical section for the design of the vertical reinforcement in the wall is the flexural crack at the base.

The critical section for the design of the horizontal reinforcement is the inclined surface along the top of the "fan" region, which is inclined at an angle (θ) to the vertical axis. The resistance of the horizontal reinforcement that crosses this inclined surface is equal to the steel contribution (V_s). In the plastic hinge region of a shear wall, the angle (θ) depends on the magnitude of axial compression applied to the wall.

The concrete contribution (V_c) that can be added to the steel contribution (V_s) depends on the width of the diagonal cracks in the plastic hinge region. The level of damage in the plastic hinge region, including the width of the diagonal crack, is related to the inelastic rotational demands. The maximum shear resistance ($V_{r,max}$) to avoid diagonal crushing of concrete is also related to the inelastic rotational demands in the plastic hinge region. The calculation inelastic rotational demand is discussed in Section 3.4.4.6 Ductility of Walls.

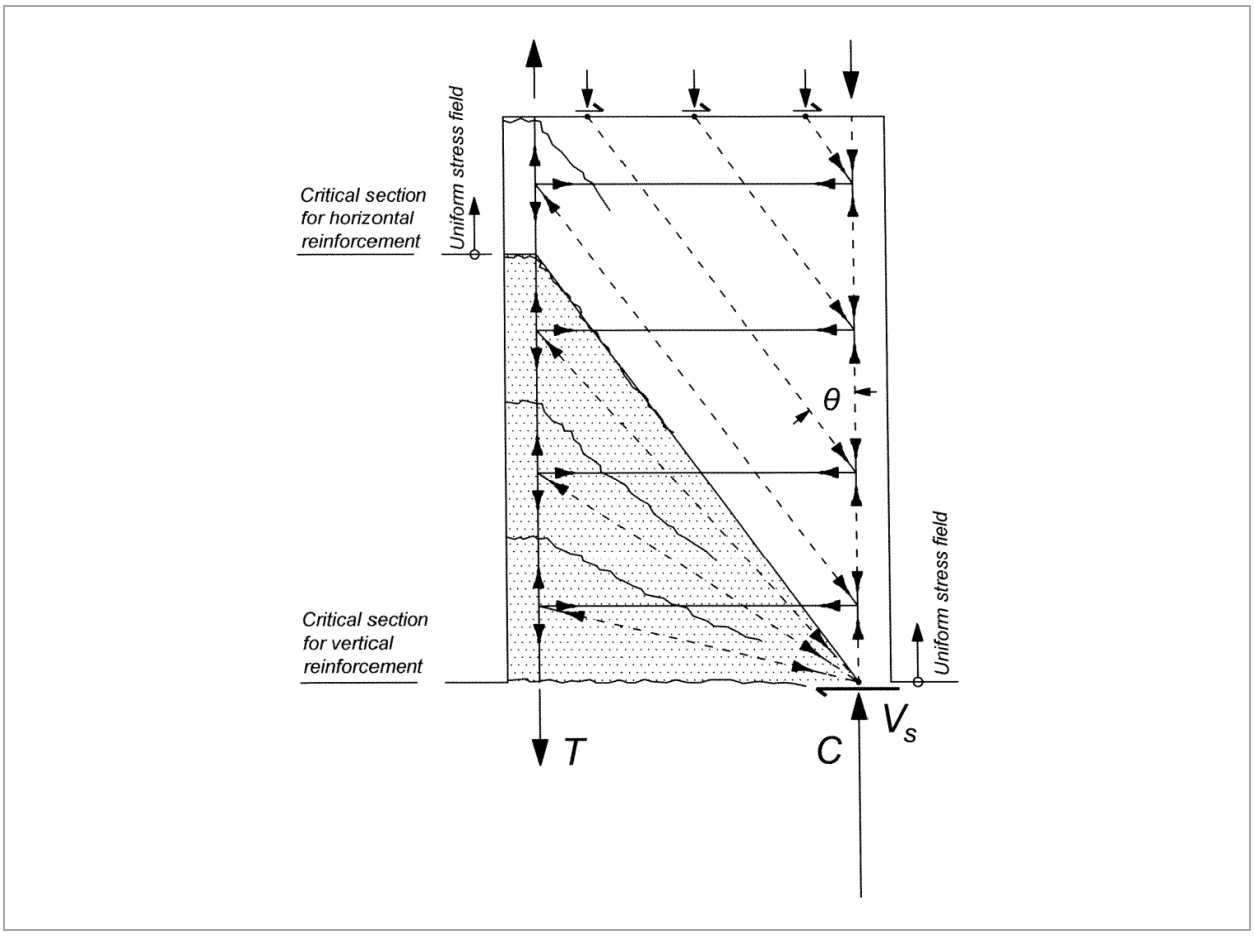


Figure 3: Strut-and-tie model describing the force flow associated with the shear resisted by horizontal reinforcement V_s at the base of a shear wall (from Adebar 2018)

Wall Openings in Plastic Hinge Region

Figure 3 above depicts how the diagonal compression stresses flow towards the flexural compression zone on the right-hand side of the cantilever shear wall. If an opening in the wall was placed in the "path" of these diagonal compression struts, a shear failure could result before the flexural capacity of the wall is reached. Naturally, the critical zone occurs on the opposite side of the wall when the lateral force reverses direction (i.e., the shear force is towards the left).

In many buildings, the "critical section for vertical reinforcement" (also referred to as the "base of the plastic hinge") shown in Figure 3 occurs at grade level.

It is often architecturally desirable and common in many buildings for an additional opening to be added to the core at the main level for the below-grade parking stairs, which needs to be a separate fire compartment from the tower scissor stairs. However, the addition of an opening at the base of the plastic hinge is structurally undesirable because it will interrupt the critical diagonal compression struts that resist the shear force. The SER should consider suggesting alternative layouts to the Architect that meet the architectural requirements without compromising or complicating the structure. Two possible alternatives are:

1. to have the parking stairs within the core stop below grade (i.e., below the critical section) and have separate stairs outside the core leading up to the exit at grade; and
2. to have the scissor stairs that go down from the tower above stop at level 2, and have the exit corridor and stairs to grade continue outside of the core.

If the Architect and SER opt for an opening in the cantilever wall at the critical section near the base of the wall, where first yielding of the vertical reinforcement in a wall is expected to occur, the SER must pay particular attention to the shear design, which is critical and typically governs the design in this region.

Similar to what is discussed in Section 3.4.4.3 Design of Wall Piers for Axial Load and Bending Moment regarding the effect of wall openings on the flexural resistance of wall piers, CSA A23.3 states that the properties of the wall cross section that affect the shear resistance of the wall must be maintained over the height of the plastic hinge region, suggesting that a wall opening that affects the shear resistance is not permitted. CSA A23.3-19 has revised and clarified this requirement such that the shear resistance must be maintained over the height of the plastic hinge region, and the effect of wall openings must be accounted for.

Strut-and-tie models may be used to confirm that the diagonal compression due to shear forces in a wall can be transmitted around small openings and may be used to determine the additional reinforcement required around an opening.

3.4.4.6 Ductility of Walls

The ductility provisions in CSA A23.3 ensure that the concrete shear walls will not suffer severe damage due to crushing of concrete in compression or fracture of reinforcement in tension during the design level ground shaking.

CSA A23.3 requires the calculation of inelastic rotational demands to be compared with inelastic rotational capacities. The inelastic rotational demands are determined from the top wall displacements, which are determined from the Linear Dynamic Analysis of the building using the effective stiffness values given in CSA A23.3. The effective stiffness is a function of the wall (nominal) overstrength, which is not known until the design of the reinforcement is completed. Thus, the evaluation of ductility is usually done using an initial estimate of top wall displacement based on the minimum overstrength, which results in a lower-bound estimate of effective stiffness and upper-bound estimate of wall displacement. If the ductility requirements are not satisfied (i.e., the inelastic rotational demand exceeds inelastic rotational capacity), refined estimates of effective stiffness should be determined after the reinforcement design is completed in order to make reduced estimates of inelastic rotational demands.

The inelastic rotational demands and capacities need to be evaluated in the plastic hinge region near the base of the core, separately in the cantilever-wall and coupled-wall directions, to limit damage in the wall piers. Additionally, the inelastic rotational demands and capacities of the coupling beams need to be evaluated to limit damage in the coupling beams.

The CSA A23.3 limit of 0.04 for the inelastic rotational demands on diagonally reinforced coupling beams is often the governing ductility requirement. If the requirements are not satisfied, a refined estimate of building displacement should be made in order to reduce the calculated rotational demands. If that is not sufficient, the building design must be modified. The building displacements (and hence the coupling beam rotational demands) can be reduced by increasing the strength of the coupled wall system and/or changing the core geometry. Section 3.4.4.4 Factored, Nominal, and Probable Flexural Resistances of Core Walls describes how the quantity of vertical reinforcement in the wall piers influences the nominal overstrength of a coupled wall system. Increasing the length of the coupling beams in order to reduce the coupling beam rotation for a given building displacement may not be effective because the increased coupling beam lengths may result in larger building displacements.

Traditionally, confinement reinforcement (as defined in CSA A23.3) has not been provided in concrete wall piers in BC, and thus the concrete compression strains must be limited to 0.0035. The neutral axis depth (c) that is used to determine the inelastic rotational capacity of the wall pier from the maximum compression strain is calculated using the factored compression strength of concrete ($\phi_c f'_c$). This low level of compression stress compensates for not accounting for the variation of compression stress (and compression strain) across the "flange" of a concrete wall pier due to shear lag effect in the calculation of the neutral axis depth (c).

In addition to ensuring a concrete wall has adequate ductility at all the expected plastic hinge locations, CSA A23.3 requires a check on the compression strain depth to ensure the wall has adequate ductility to tolerate limited yielding of vertical reinforcement due to higher mode bending moments at any point over the height of the wall.

3.4.4.7 Seismic Detailing Requirements

CSA A23.3 specifies a number of detailing requirements that are intended to ensure concrete walls will maintain their strength and displacement capacity while subjected to reverse cyclic demands during an earthquake. Most of these requirements relate to the amount and arrangement of reinforcement; however, there is also a requirement for the minimum wall thickness to prevent buckling failures of wall piers.

The following is a summary of notable detailing requirements in CSA A23.3:

- **Wall thickness**: The minimum wall thickness is specified as a ratio of the unsupported length of the wall. Usually, this is the unsupported vertical height of wall between floor slabs which provide horizontal support to the wall. If the core is located at the perimeter of the building, it is possible to have a wall between the elevator shaft and the outside of the building that is not supported by any floor slabs. In that case, the unsupported length of the wall is taken as the maximum unsupported horizontal length of wall between two or more lateral supports. A larger minimum wall thickness is required in the plastic hinge region of the wall.
- **Reinforcement splices**: CSA A23.3 specifies increased lap splice lengths, describes limitations on the use of mechanical splices, and prohibits reinforcement being offset bent. For ductile wall systems ($R_d \geq 3.5$), not more than 50% of the reinforcement at each end of the walls in plastic hinge regions can be spliced at the same storey level, and at least one-half of the storey height must be completely clear of lap splices in the concentrated reinforcement.

- **Distributed reinforcement**: The minimum percentage of distributed vertical and horizontal reinforcement over the full height of the wall is increased, and the maximum spacing of the horizontal distributed reinforcement in the plastic hinge region is reduced.
- **Anchorage of horizontal reinforcement**: There are special requirements for the anchorage of horizontal reinforcement at the ends of the wall over the full height of the wall, and there are additional requirements in the plastic hinge region.
- **Tied concentrated reinforcement**: Specially tied concentrated vertical reinforcement must be provided at the ends of all walls over the full height of the walls. The closed ties around the perimeter of the concentrated reinforcement and any crossties must have special seismic hooks that are anchored into the "confined core" within the vertical reinforcement.
- **Tie spacing**: There are two different maximum spacings of the ties on the concentrated reinforcement.
  - The wider spacing is the standard spacing of ties on (non-seismic) compression members, usually limited to 16 times the diameter of the smallest vertical reinforcing bar or the least dimension of the member. This spacing is used outside the plastic hinge region, at the wall ends not connected to coupling beams.
  - Over the height of the plastic hinge region, and over the full height of ductile (R_d = 4.0) walls that are connected to coupling beams, the spacing of the ties must be reduced to prevent buckling of the vertical reinforcement under reverse cyclic loading. The ties spacings are usually limited to the smaller of 6 times the diameter of the smallest vertical reinforcing bar or one-half of the least dimension of the member.
- **Additional vertical reinforcement below the plastic hinge region**: Additional vertical reinforcement must be added below the plastic hinge region in the seismic force resisting walls. The portion of wall immediately below the critical section at the base must contain a minimum of 20% additional vertical reinforcement than the wall immediately above the critical section.
- **Gravity-load resisting members**: CSA A23.3 requires that the more closely spaced ties for concentrated reinforcement that prevent vertical reinforcing bar buckling also be provided in gravity-load columns over the height that the SFRS is required to be detailed for plastic hinging to occur (i.e., the plastic hinge region). All gravity-load walls must also have tied concentrated vertical reinforcement at each end of the wall and at the ends and intersections of all wall "flanges" over the plastic hinge region. The wider spacing of the ties may be used in the gravity-load walls.

3.4.4.8 Transfer of Horizontal Wall Forces Across Construction Joints

Sliding shear failures during earthquakes have been observed when construction joints are not properly cleaned and roughened. To prevent a sliding shear failure, the interface shear resistance of the construction joint must be equal to or greater than the seismic shear force applied to the wall pier.

CSA A23.3-14 provides general procedures for determining the interface shear resistance that can be applied to construction joints, but provides limited guidance on how the procedures are applied to construction joints in walls. The limited guidance provided in CSA A23.3-14, Clause 14.1.6 was removed in CSA A23.3:19 because of concern that it may not always be appropriate. The commentary to Clause 21.5.9.1 in the *Concrete Design Handbook* (Cement Association of Canada 2015) provides additional information on how to calculate the interface shear resistance of construction joints.

3.4.5 REFINED ANALYSIS OF STRUCTURE BELOW PLASTIC HINGE ZONE

When the concrete (tower) shear walls are connected by multiple floor diaphragms to other walls, such as perimeter foundation walls or podium walls, the plastic hinge in the tower walls usually happens above this level. In this case, the system which resists the overturning moment and shear forces below the plastic hinge is indeterminate. The overturning moment can be transferred directly to the foundation below the tower walls, or can be transferred to the foundation walls or podium walls by force couples in two or more floor diaphragms. This second load path is sometimes referred to as the backstay effect.

When a significant portion of the overturning moment is transferred by force couples in the diaphragms, the bending moment gradient (which is equal to the shear force) in the tower walls will reverse below the flexural hinge. This may create large reverse shear forces in the tower walls that are often significantly larger than the base shear force.

To design a safe structure, the Code requires that an assessment be made to determine what portion of the forces in the tower walls may be resisted by each load path. As the assumed stiffness properties of the elements in each load path will significantly influence the forces, and these properties are difficult to determine accurately, the Code requires that the analysis consider a range of possible stiffness properties. To make sure that the design provides adequate strength in the structural elements of each load path, multiple analyses with upper-bound or lower-bound stiffness properties of the elements must be done to determine the maximum forces in each load path.

In typical practice in the United States, this type of analysis is done as part of the response history analysis of the entire building. CSA A23.3 permits a simplified static analysis of only the structure below the plastic hinge. Multiple static analyses (with different load cases based on the capacity of the plastic hinge in the tower walls) are used in lieu of a dynamic analysis (with varying forces) to determine the envelope of design forces.

Four different analysis cases with different loading and different member stiffnesses must be done to determine the following information for each case:

1. Maximum bending moments in tower walls below the plastic hinge, and design forces for foundation supporting the tower walls.
2. Design forces from the second load path, where the overturning moment is transferred to the foundation walls or podium walls by force couples in two or more floor diaphragms:
   (a) forces in the diaphragms, other walls, and the associated connections; and
   (b) maximum (reverse) shear force in tower walls.
3. Interstorey drift ratio of tower walls at top of the subterranean or podium structure.

Case 1 provides the design forces from the overturning moment being transferred directly to the foundation below the tower walls. Thus, an upper-bound estimate is used for the flexural rigidity of the tower walls and the stiffness of the soil or rock below the foundation, and lower-bound estimates of stiffness are used for the other actions and members (shear rigidity of tower walls, stiffness of diaphragms, and stiffness of other walls). The forces applied to the structure below the plastic hinge for case 1 is the nominal bending moment capacity of tower walls and the associated shear force.

Cases 2(a) and 2(b) provide the design forces from the second load path, where the overturning moment is transferred to the foundation walls or podium walls by force couples in two or more floor diaphragms. A lower-bound estimate is used for the flexural rigidity of the tower walls; the flexibility of the soil or rock below the foundation must be accounted for (as required by the Code); and upper-bound estimates of stiffness are used for the other actions and members. The forces applied to the structure below the plastic hinge for case 2(a) is the probable bending moment capacity of tower walls and the associated shear force, while for case 2(b), it is also the probable bending moment capacity of tower walls but with a zero-shear force. The case of zero-shear is needed to get the maximum estimate of

reverse shear force in the tower walls accounting for higher mode shears using a static analysis.

Case 3 provides the additional deformations of the base accounting for foundation movement and cracking of concrete in the subterranean structure. The inter-storey drift ratio of the tower walls at the top of the subterranean or podium structure must be added to the deformations determined in the global model using a simplified model of the subterranean structure, as discussed in Section 3.4.3.1 Modelling Requirements. Estimates based on the best available information should be used for all member stiffnesses.

While it is known whether an upper-bound or lower-bound stiffness is safe for each member, the range of possible stiffness values is often too large not to refine the estimate of stiffness based on the level of load determined in the analysis. For example, the effective flexural rigidity and effective shear rigidity of the concrete diaphragms range from 100% of the uncracked rigidities at low load levels, to as low as 5% of the uncracked section rigidities at the onset of reinforcement yielding (Adebar and Mahmoodi 2020a). Given the large impact the assumed stiffness can have on the design forces, the SER should consider conducting additional static analyses with refined estimates of upper-bound or lower-bound stiffness, to reduce the design forces.

Additional information on how to conduct the static analyses is given in the commentary to CSA A23.3-14, Clause 21.5.2.2.9, and an example is given in Chapter 11 of the *Concrete Design Handbook* (Canadian Concrete Association 2015).

3.4.6 DESIGN OF GRAVITY-LOAD RESISTING FRAMES FOR SEISMIC DEFORMATION DEMANDS

3.4.6.1 Overview of Requirements

Perhaps the most significant change to the Code in the current edition (*BCBC 2018* and VBBL 2019) is the additional requirements for the seismic design of the Gravity-Load Resisting Frames. While these members are not part of the SFRS, they are connected to the SFRS by the floor slabs, and thus will be subjected to the same seismically induced deformations. Observations from past earthquakes have shown that the collapse of concrete buildings is often triggered by failure of columns or walls in the Gravity-Load Resisting Frame.

Gravity-Load Resisting Frame members either must be flexible enough to tolerate Elastic Deformation, where the members experience displacements while remaining elastic (undamaged), or must be detailed to be sufficiently ductile to tolerate Inelastic Deformation while continuing to support the gravity loads.

The requirements to prevent collapse of the Gravity-Load Resisting Frames (CSA A23.3, Clause 21.11) do not have to be applied when the seismic displacements are expected to be small. This is the case for any building in regions of low seismicity and for stiff low-rise buildings in all regions. However, the provisions are critical for tall concrete buildings in the Lower Mainland of BC and on Vancouver Island, as large displacements are expected in these buildings.

As the concrete shear walls in a tall concrete building deform during an earthquake, they will induce force and displacement demands into the surrounding Gravity-Load Resisting Frame in a number of different ways. The most general approach to determining the demands on the Gravity-Load Resisting Frame is response history analysis using a nonlinear model of the complete building; however, this is rarely done. A more practical approach is to use a nonlinear model of the SFRS and a linear model with appropriate effective stiffness properties for the Gravity-Load

Resisting Frame. As Non-linear Dynamic Analysis is rarely used for the design of tall concrete buildings in BC, further simplified procedures are provided in the Code to estimate the demands on the Gravity-Load Resisting Frame due to the nonlinear response of the SFRS.

Linear Dynamic Analysis of a tall concrete shear wall building using the effective stiffness properties of the wall (CSA A23.3, Clause 21.2.5.2) can be used to make a good estimate of the maximum displacement at the top of the wall accounting for non-linear response of the concrete walls. While applying the average reduced flexural rigidity ($E_c I_e$) of the shear walls over the full height of the shear walls gives a good estimate of the top wall displacement, it does not give a good estimate of distribution of deformations over the height of the building. Yielding in the plastic hinge region near the base of the building causes a concentration of the bending deformation at that level, which is not accounted for in Linear Dynamic Analysis.

CSA A23.3, Clause 21.11 includes a number of simplified solutions for converting the estimate of top wall displacement to the deformation demands on the Gravity-Load Resisting Frame members at various elevations. These solutions can be used to determine critical deformation demands on the Gravity-Load Resisting Frame without the need to do a complete analysis of the Gravity-Load Resisting Frame.

The critical deformation demands include:

- bending demands on gravity-load resisting columns and walls in the critical plastic hinge region near the base of the structure;
- deformation demands on slab–column connections (degrading punching shear resistance) over the full height of the building;
- bending demands on gravity-load resisting columns and walls at critical levels; and
- increased axial load demands on gravity-load resisting columns and bearing walls at critical levels.

3.4.6.2 Thin Concrete Bearing Walls

Thin concrete walls can be a cost-effective structural component; for example, as partition walls in a low-rise building where the axial load on the wall is low and the building does not experience significant interstorey drift.

Thin concrete walls were commonly used as both gravity-load resisting elements (bearing walls) and lateral load resisting elements (shear walls) in tall concrete building in BC prior to the mid-1980s (Adebar, DeVall, and Mutrie 2017). It is now well known that special attention is needed whenever a thin concrete wall, or any gravity-load resisting wall, is included in a tall concrete building (Adebar 2013). The axial load must be kept low as heavily loaded thin concrete walls can very suddenly lose all axial load carrying capacity. When concrete walls are used as lightly loaded partition walls in the lower levels of a tall concrete building, they can inadvertently attract significant seismic shear forces and/or add significant torsional eccentricity to the building.

To mitigate the effects and concerns related to thin concrete walls, a number of significant changes were made to CSA A23.3. The following are some of the notable changes:

- The maximum factored axial load resistance $P_{r,max}$ of walls with a thickness less than 300 mm has been reduced by up to 45% over previous editions of the standard.
- Thin bearing walls with a single layer of reinforcement cannot be used to resist any gravity loads in a building where the interstorey drift ratio (including the influence of accidental torsion) exceeds 0.005 (0.5%) at any point in the building.
- When assessing the influence of bending moments induced into gravity-load resisting walls due to seismic deformations, a very low calculated bending moment is permitted in thin walls.
- Strong axis bending of bearing walls due to lateral deformation of the building due to wind or earthquake must be accounted for.
- Slenderness effects must be account for in the compression zone of walls subjected to strong axis bending.

3.4.6.3 Design of Gravity-Load Columns and Walls in the Plastic Hinge Region

While the interstorey drift ratios are generally smallest near the base of a shear wall building, the rate of change of interstorey drift ratios is maximum in this zone due to the concentrated flexural deformations in the plastic hinges in the concrete shear walls. The rate of change of interstorey drifts ratios (rate of slope change per unit height) is equal to the curvature.

The gravity columns and walls are tied to the shear walls by multiple floor slabs that have very high in-plane stiffness. The floor slabs force the gravity-load columns and walls to have the same deflected shape and hence the same curvature distribution as the shear walls. Thus, the maximum curvature demands on the gravity-load columns and walls over the height of the plastic hinge region of the building can be determined without the need for an analysis model of the Gravity-Load Resisting Frame.

As discussed in Section 3.4.4.6 Ductility of Walls, CSA A23.3 requires the calculation of inelastic rotational demands on the concrete shear walls. This information can be used to determine the maximum curvature demands on the gravity-load resisting columns and walls.

Over the storeys that the SFRS is required to be detailed for plastic hinging to occur, CSA A23.3 requires that all columns and walls must have a curvature capacity greater than the curvature demand on the concrete shear walls in that direction. Specifically, as discussed in Section 3.4.4.7 Seismic Detailing Requirements, CSA A23.3 requires that tied concentrated reinforcement be provided at the ends of gravity-load resisting walls, and anti-buckling ties be provided in all gravity-load resisting columns in this region.

Above the plastic hinge region, the gravity-load resisting columns and walls are subjected to bending demands because of a different phenomenon than happens in the plastic hinge region. This is discussed further in Section 3.4.6.6 Bending Demands on Gravity-Load Columns and Walls.

3.4.6.4 Distribution of Seismic Demands Over Building Height

CSA A23.3 provides a simplified solution for the distribution of seismic demands over the building height that can be used in lieu of conducting an analysis of the Gravity-Load Resisting Frame.

As described above, Linear Dynamic Analysis with the effective stiffness properties of the wall defined in CSA A23.3, Clause 21.2.5.2 is used to determine the maximum displacement at the top of the walls accounting for non-linear response of the concrete walls. The influence of inherent and accidental torsion must be included in the analysis. The additional interstorey drift ratio calculated at the top of the subterranean or podium level accounting for foundation movement and cracking of concrete in the subterranean structure, as discussed in case 3 of Section 3.4.5 Refined Analysis of Structure Below Plastic Hinge Zone, must be added.

The envelope of top wall displacements is used to calculate the maximum top displacement of each Gravity-Load Resisting Frame. A Gravity-Load Resisting Frame may consist of a single column or bearing wall connected to a shear wall by slabs (or beams), or may consist of many interconnected columns and/or walls. There are typically several different Gravity-Load Resisting Frames in each direction of a building, and all Gravity-Load Resisting Frames must be investigated in each direction of loading. Twisting of a building due to inherent and accidental torsion causes parallel Gravity-Load Resisting Frames in one direction of a building to have different maximum top displacements.

CSA A23.3 defines the envelope of interstorey drift ratios that relates the top displacement of the Gravity-Load Resisting Frames to the deformation profile over the height of the building accounting for the non-linearity of the concrete shear walls. Figure 21.1 in CSA A23.3-14 provides the envelope to be used in the cantilever wall direction of shear wall buildings, while the envelope in the coupled wall direction is given in Figure N21.11.2.2 (a) of the commentary to CSA A23.3-14, or Figure 21.1 in CSA A23.3-19.

See Figure 4 below for envelopes of interstorey drift ratios that relate top displacement of Gravity-Load Resisting Frames to deformation profile over the height of shear wall buildings.

The envelope of interstorey drift ratios, called "building interstorey drift ratios," are equal to "the difference in lateral displacement of the building per storey" divided by "the storey height." How the building drifts influence the demands on the members of the Gravity-Load Resisting Frame depends on the relative bending stiffness of the floors and the columns/walls.

When assessing the influence of seismic displacement demands on the punching shear resistance of flat-plate slabs, the flexibility of the columns can be ignored (column drift is assumed to be zero) for simplicity.

The check for punching shear failure is discussed further below.

When assessing the influence of seismic displacement demands on the gravity-load columns and bearing walls, the building interstory drift ratio must first be converted to the column drift ratio (δ_{col}), which is dependent on the relative stiffness of the frame members. The conversion can be done using a simple structural analysis model for one level (or a few levels) of the Gravity-Load Resisting Frame, or can be estimated using the interaction diagram presented in Figure N21.11.2.2 (b) in the commentary to CSA A23.3. The interaction diagram summarizes the analysis results from a range of different Gravity-Load Resisting Frames (Adebar, DeVall, and Mutrie 2014).

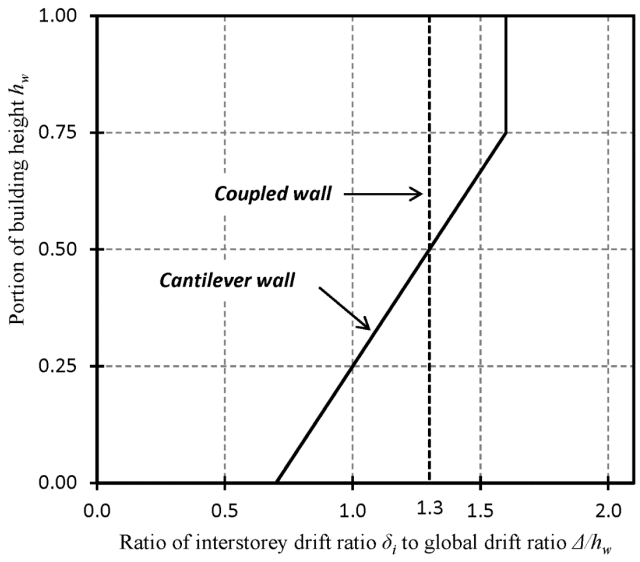


Figure 4: Envelopes of interstorey drift ratios that relate top displacement of Gravity-Load Resisting Frames to deformation profile over the height of shear wall buildings

Effective Stiffness of Gravity-Load Resisting Frame Members

Except for a few critical members, CSA A23.3 does not specify the effective stiffness to be used for the Gravity-Load Resisting Frame members. The SER must use professional judgment when conducting the analysis of the Gravity-Load Resisting Frame.

CSA A23.3 defines lower-bound estimates of effective stiffness for the SFRS to make a safe estimate of the design displacement; but upper-bound estimates of effective stiffness must be used for the Gravity-Load Resisting Frame members to make safe estimates of the forces induced in these members by the Inelastic Deformation (displacement) profile of the SFRS.

Because of concerns with walls, particularly thin walls, the uncracked section flexural rigidity ($E_c I_e = 1.0 E_c I_g$) must be used for both the strong-axis and weak-axis bending of these members, unless they contain compression ties over the full length (CSA A23.3, Clause 21.11.3.3.3).

It is recommended that the SER use a bending moment–curvature analysis (sometimes called fibre model analysis) of the Gravity-Load Resisting Frame members to determine an appropriate effective stiffness.

3.4.6.5 Slab-Column Connections

CSA A23.3 specifies the reduction in punching shear resistance of concrete slabs due to the building interstorey drift ratio. The punching shear resistance reduces at interstorey drift ratios larger than 0.005 (0.5%). The load combination used to calculate the punching shear stress for the earthquake load combination is $1.0D + 0.5L$, while the punching shear resistance without a reduction due to interstorey drifts from earthquake must be sufficient to resist the gravity load combination, $1.25D + 1.5L$. Thus, the reduction in punching shear resistance typically does not require shear reinforcement be added in the slab until interstorey drift ratios of about 0.01 (1.0%). See Section 3.4.2.3 Gravity-Load Resisting Frame for further discussion.

The envelope of building interstorey drift ratios defined by CSA A23.3, including the effect of inherent and accidental torsion and the additional interstorey drift ratios calculated at the top of the subterranean or podium level accounting for foundation movement and cracking of concrete in the subterranean structure, must be used to evaluate the punching shear resistance of slab-column connections in each Gravity-Load Resisting Frame.

3.4.6.6 Bending Demands on Gravity-Load Columns and Walls

Shear walls apply bending demands to the gravity-load resisting columns and walls through the connecting floors in two different ways. First, due to the very high in-plane rigidity of the closely spaced floor slabs, the gravity-load columns and walls are forced to have the same deflected shape as the shear walls. Throughout most of the height of the building, the curvature demands in the shear walls are low compared to the curvature capacity of the gravity-load columns and walls. This is not the case, however, in the plastic hinge region of the shear walls. As discussed in Section 3.4.6.3 Design of Gravity-Load Columns and Walls in the Plastic Hinge Region, the curvature demands in the shear walls cause large bending demands on the gravity-load columns and walls in the plastic hinge region.

The second way that shear walls impose bending demands on the gravity-load resisting columns and walls is through frame action resulting from building drift, which is larger in the upper parts of the building. As described above, when assessing the bending demands on gravity-load columns and bearing walls from frame action, the building interstorey drift ratio must first be converted to column drift ratio (δ_{col}) using either a simple structural analysis model for one level (or a few levels) of the Gravity-Load Resisting Frame, or the interaction diagram presented in the commentary to CSA A23.3.

When the Gravity-Load Resisting Frame consists of relatively long-span flat-plate slabs, the column drifts tend to be a small portion of the building drifts. That is,

very little bending is induced into the columns or walls. On the other hand, when a deep transfer girder or transfer slab connects to a gravity-load column or bearing wall, very large column bending can be induced into the member.

For most buildings, it is not necessary to determine the bending moment induced into every gravity-load resisting column and bearing wall at every level; but rather to spot check a few cases where the induced bending moments are expected to be larger. Any member supporting a thicker floor member (e.g., transfer or shorter span floor member) should be checked. For a uniform building with uniform slab thicknesses, uniform slab spans, and uniform column dimensions over the building height, the maximum bending moment induced in the column will be at the top floor (roof) level, because there is only one column framing into the slab at that level, making the relative floor-to-column stiffness twice as large as on any other floor level.

The column lateral displacement per storey height (drift ratio, δ_{col}) can be converted to an induced bending moment in the column or wall using the relationship:

$$M = {}^{6\,E_c I_e}/_{h_s} \cdot \delta_{col},$$

where $E_c I_e$ is the flexural rigidity of the column or wall in the direction of bending; and
h_s is the storey height.

CSA A23.3 limits the maximum calculated induced bending moment based on whether the member is a column or wall, the level of axial compression applied to the member, and the reinforcement arrangement.

If the calculated induced bending moment is larger than the CSA A23.3 limit, the design of the Gravity-Load Resisting Frame must be modified. The simplest solution is often to increase the ductile detailing in the column or wall to permit higher levels of damage in the member. However, if the bending moment induced in the column or wall is too large, the geometry of the Gravity-Load Resisting Frame members must be modified to reduce the induced bending moment. An example for how the design of very deep transfer girders can be modified to reduce the induced bending moment is given in Section 3.4.6.7 Increased Axial Compression in Gravity-Load Columns.

3.4.6.7 Increased Axial Compression in Gravity-Load Columns

In addition to the bending moments induced in the gravity-load columns and walls, the lateral building displacements due to earthquake motions will also cause horizontal members such as slabs or beams to induce additional vertical load into supporting columns and walls. This phenomenon is sometimes referred to as the "outrigger effect."

For typical buildings with relatively long-span flat-plate slabs, the additional vertical forces due to earthquake displacements typically do not result in a governing load combination for the column or wall design, as the load combination $1.0E + 1.0D + 0.5L$ results in a lower factored design force than the load combinations $1.25D + 1.5L + 1.0S$ or $1.4D$.

If the slab has a short span over many storeys because the gravity-load columns or walls are located close to the core, or are located close together, significant axial forces may accumulate over the height of the building due to the outrigger effect. In that case, an analysis must be done to determine the magnitude of the axial force by summing the contribution from all levels above the level of interest. Minimum recommended clear spans when outrigger effects are not explicitly modelled are 6 m for column-to-core and 3 m for column-to-column (LATBSDC guidelines).

For most buildings and similar bending moments, it is sufficient to spot check a few cases where the induced vertical forces in the supporting members are expected to be larger. Any member supporting a thicker member (e.g., transfer or shorter span floor member) should be checked. The shear force in the horizontal member (slab or transfer girder) can be determined from a simple structural analysis model for one level (or a few levels) of the Gravity-Load Resisting Frame, using the CSA A23.3-defined building interstorey drift ratio for that level.

The additional vertical force due to seismic deformations does not need to be taken greater than the maximum shear force that can develop due to the nominal flexural resistance of the attached horizontal members. That is, flexural yielding of the horizontal member limits the maximum shear force in that member.

If the calculated induced vertical load in the supporting column or wall is too large, the design of the Gravity-Load Resisting Frame must be modified by reducing either the bending stiffness or the bending strength of the horizontal member. An example of how the SER of the Living Shangri-La building in Vancouver, BC achieved this is outlined below.

The Living Shangri-La building, which is currently the tallest building in BC at 201 m high, has gravity-load columns that start at the top of the building and are transferred near grade level on 3.6-m-deep transfer girders that are supported at one end by the core and the other end by columns. The very deep transfer girders naturally would act as outriggers when the core bends laterally and would overload the gravity-load columns supporting the other end of the transfer girders. To mitigate against this, the 3.6-m-deep transfer girders were cast monolithically with the core walls over only the lower 0.9 m of depth, and a gap was left over the remaining 2.5 m height of the transfer girders. That is, the transfer girders were designed with a pin support to the core walls (Adebar, DeVall and Mutrie 2017).

3.4.6.8 Complete Analysis of Gravity-Load Resisting Frame

As described above, two of the four critical demands on Gravity-Load Resisting Frames described in Section 3.4.6.1 Overview of Requirements can be determined without an analysis of the Gravity-Load Resisting Frame. These are the critical bending demands on gravity-load resisting columns and walls in the plastic hinge region near the base of the structure, and the deformation demands on slab-column connections over the full height of the building.

The bending demands and increased axial load demands on gravity-load resisting columns and walls at particular can be determined from the envelope of building interstorey drift ratios using a simple structural analysis model for one level (or a few levels), or using the interaction diagram presented in the commentary to CSA A23.3. The main challenge is selecting the appropriate model and effective stiffness for the critical members of the Gravity-Load Resisting Frame. An example is the challenge of accurately modelling outrigger slabs accounting for the torsional flexibility of the slabs and the level of cracking commensurate with the lateral displacements of the building. The recommended approach is to spot check critical points in the Gravity-Load Resisting Frame rather than analyze the complete frame. One reason for this is that it is expected that there will be only a few "hot spots" where the Gravity-Load Resisting Frame will need to be modified to reduce induced demands, rather than widespread changes.

There are challenges with using a model of the complete building to determine the bending demands and increased axial load demands on gravity-load resisting columns and walls without doing a Non-linear Dynamic Analysis of the building.

The profile of displacements over the height of the building must reflect the concentration of deformations at the plastic hinge locations in the concrete shear walls (the profile of displacements will not be linear). In lieu of using a non-linear model of the concrete shear walls, a linear model with appropriately reduced section properties at plastic hinge locations may be used to estimate the Inelastic Deformation (displacement) profile of the building. The challenge with this relatively simple approach is having the correct length over which the section properties are reduced, and having the correct relative reduction in section properties to yield the correct distribution of deformations, as will occur in the non-linear structure.

CSA A23.3 defines the height of the plastic hinge region over which special detailing must be provided; however, the inelastic curvatures are not uniform over this height. To simulate the inelastic curvatures in the

plastic hinge region, the height of wall with reduced section properties must be half the height of the plastic hinge region defined by CSA A23.3.

The envelope of interstorey drift ratios over the height of the building results from a number of different modes of the SFRS. The maximum interstorey drift ratios near the top of the building result from the first mode, while the maximum interstorey drift ratios near the mid-height of the building result from higher modes. If a static analysis is used, multiple analyses with different force distributions are required to determine the envelope of interstorey drift ratios.

Shear Strains in Concrete Walls

The additional interstorey drifts resulting from shear strains in the plastic hinge regions of the concrete walls must be accounted for. This complex phenomenon is not accounted for in most non-linear analysis programs but is included in the envelope of interstorey drift ratios defined by CSA A23.3.

The shear strains are the result of vertical tension strains due to flexural yielding of the vertical reinforcement in walls with diagonal cracks. The magnitude of the shear strain, which equals the magnitude of the resulting interstorey drift ratio, can be reasonably estimated from inelastic rotational demands on the wall (Adebar 2018). Over the height of the plastic hinge region of concrete walls, the interstorey drift ratio must not be taken less than 60% of the inelastic rotational demand (i.e., $0.6\theta_{id}$).

3.4.7 ADVANCED DESIGN ISSUES

3.4.7.1 Gravity-Induced Lateral Demand Irregularity

The severity of a gravity-induced lateral demand (GILD) irregularity is quantified in the Code in terms of the ratio, α. For a shear wall core, α is equal to the "gravity-load bending moment applied to the shear wall core at the critical section where flexural yielding will first occur" to the "bending moment resistance of the shear wall core." The limits on α depends on whether the SFRS has self-centring characteristics or not. A cantilever shear wall subjected to significant axial compression would be considered self-centring, while a coupled wall system typically would not.

When α is small, the effect of the GILD can be ignored. When α is larger (Table 2 below), the displacements of the building must be increased by 20% to account for the "ratcheting" of the displacements in the direction of the GILD. When α is beyond a certain limit (Table 2), Non-linear Dynamic Analysis must be used to demonstrate that the performance of the building will be acceptable.

Vertical acceleration of the mass generating the GILD will increase the severity of the irregularity. Thus, according to the *NBC 2020*, the analysis must account for the vertical response of the building mass, and the Non-linear Dynamic Analysis must include vertical ground motion time histories in addition to the horizontal ground motion time histories.

The accumulation of building deformations over several earthquake cycles and the resulting residual displacements are very difficult to predict even using the most sophisticated Non-linear Dynamic Analysis methods. To ensure adequate performance of the building, the acceptance criteria for the Non-linear Dynamic Analysis results must be adjusted from what is used for regular building performance. The largest interstorey displacement at any level is limited to 60% of 0.025 h_s (i.e., 0.015 h_s), and the other analysis results (forces and displacements) must be multiplied by 1.5 before comparing with the non-linear acceptance criteria for the building.

Table 2: Gravity-Induced Lateral Demand Severity Ratio, α

SELF-CENTRING SFRS	OTHER SFRS	REQUIREMENT
$\alpha \leq 0.1$	$\alpha \leq 0.03$	None
$0.1 < \alpha \leq 0.2$	$0.03 < \alpha \leq 0.06$	Multiply displacements by 1.2
$\alpha > 0.2$	$\alpha > 0.06$	Non-linear Dynamic Analysis required

3.4.7.2 Sloped-Column Irregularity

An architectural form that is increasingly popular for creating unique buildings involves inclining the gravity-load columns that support the floor slabs, while the shear walls in the central core of the building remain vertical.

When the sloping columns are asymmetrically arranged in a building, they create a GILD on the core (see Section 3.4.7.1 Gravity-Induced Lateral Demand Irregularity) as well as significant additional demands that need to be accounted for. These additional demands are discussed in this section.

When the sloping columns are symmetrically arranged in a building, a static analysis would suggest the columns need to be designed for the diagonal components of the vertical gravity force, and horizontal ties (to resist tension) or horizontal struts (to resist compression) may be required between the columns; but there are no demands on the core.

The results of dynamic analyses (Adebar and Mahmoodi 2020b) have shown that there are significant additional demands that must be accounted for. The differential horizontal movements at the top and bottom of the sloped columns due to seismic deformations cause vertical movements that need to be accounted for in the analysis of the Gravity-Load Resisting Frame. The differential horizontal accelerations at the top and bottom of the inclined columns cause the vertical mass supported by the gravity-load columns to accelerate vertically, thereby generating increased seismic forces (axial loads) in the gravity-load columns, increased shear forces and bending moments in the shear walls, and increased design forces in the diaphragms that connect the inclined gravity-load columns to the shear walls. The same demand increases occur in all symmetrically arranged sloped columns.

Vertical ground motions, which are not accounted for in the Code, may add to the effect and further increase the seismic design forces.

The additional seismic forces are very sensitive to the assumed stiffnesses of the columns and supporting members. The maximum design forces occur when the first vertical mode is coupled with a lateral mode of the shear walls, typically the second mode. Since the effective stiffness of concrete structures is difficult to determine accurately, a range of stiffness values must be used to make a safe estimate of the dynamic design forces due to sloped columns.

For typical ground motions in Vancouver, BC, the vertical components of the design forces applied to gravity-load columns have been found to be up to 3.0 times the forces resulting from the weight supported by the gravity-load columns (Adebar and Mahmoodi 2020b).

NBC 2020 Requirements

The NBC 2020 includes a new type of irregularity in buildings. A “sloped-column irregularity” is considered to exist when a vertical member, inclined more than 2 degrees from the vertical, supports a portion of the weight of a structure in axial compression. Given the possible significant increase in demands due to sloped columns discussed above, the SER should consider proactively meeting these requirements if a building includes this type of irregularity.

The additional seismic design forces resulting from both the coupling of horizontal and vertical modes and the effect of vertical ground motion can be determined using Linear Dynamic Analysis and force reduction factors $R_d R_o$ = 1.0, and using a structural model that:

- accounts for the vertical acceleration of all mass supported by the inclined vertical member(s); and
- includes the SFRS, the inclined vertical member(s), and all structural framing elements that transfer inertial forces generated by the vertical acceleration of the mass supported by the inclined vertical member(s).

The additional earthquake forces are sensitive to the degree of coupling between the vertical and horizontal vibrational modes of the building; the range of possible stiffness values for all structural members must be considered to determine the maximum additional forces for design.

Further information, including a simple procedure for scaling the analysis results to avoid having to do multiple analyses with a range of stiffness values, and to avoid having to include vertical ground motions, is provided in Adebar and Mahmoodi 2020b.

3.4.7.3 Discontinuous Elements Supporting Gravity Loads

For most buildings, the effect of vertical earthquake response does not need to be considered because the gravity-load resisting elements have substantial reserve capacity in the seismic load combination; this is because the factored dead- and live-load combinations prescribed by the Code typically govern the design.

When a significant discontinuity exists in a vertical-load-carrying element, such as numerous floors of gravity-load column supported on a transfer girder, the vertical building response can significantly amplify the demands. Section 3.4.8.2 Modelling Considerations below describes how vertical ground motions must be included in the evaluation of life safety performance using Non-linear Dynamic Analysis.

Currently, there are no requirements in the Code to deal with this matter when modal response spectrum method is used to determine the seismic demands; but it is expected that additional requirements will be added in future editions of the Code (based on the *NBC 2025*).

When a significant discontinuity exists in a vertical-load-carrying element, the SER should consider including the vertical response in the modal response spectrum analysis of the building. The vertical mass must be included in the model with sufficient accuracy in horizontal distribution to determine the numerous vertical modes of response correctly. The Structural Commentaries to the *NBC 2020* provides guidance on the level of vertical acceleration that should be included in the analysis.

3.4.8 EVALUATION OF LIFE SAFETY PERFORMANCE USING NON-LINEAR DYNAMIC ANALYSIS

3.4.8.1 Introduction

The Code permits Non-linear Dynamic Analysis to be used to determine the seismic demands on a building; however, as a "special study shall be performed," which triggers additional requirements, it has not been common practice to do so.

The Code requires the use of Non-linear Dynamic Analysis to evaluate the performance of a building with a large GILD irregularity.

Evaluating the life safety performance of a building using Non-linear Dynamic Analysis is particularly useful when a building has a new type of significant irregularity that is not one of the ten irregularities currently identified in the Code. Such an irregularity is referred to as an unusual irregularity.

One common misconception is that Non-linear Dynamic Analysis can be used to reduce the design requirements, compared to the prescriptive procedures in the Code. That is not the case. The intent of using Non-linear Dynamic Analysis is to extrapolate the Code-prescriptive requirements to something different from what the

Code was written for. When applied correctly, life safety performance evaluation using Non-linear Dynamic Analysis should give the same result as the Code-simplified procedures for the type of building for which the simplified procedures were developed.

Peer Review

Whenever Non-linear Dynamic Analysis is used to justify the seismic design of a concrete building, the Code requires that "the non-linear analysis and resulting design shall be reviewed by a qualified independent review panel." This mandatory requirement is clearly stated in CSA A23.3, Clause 21.2.3 and is commonly referred to in the industry as a Peer Review. Refer to Section 4.1.7 Documented Independent Review of Structural Designs and Section 4.3 Peer Review for more information.

The Peer Review panel must approve the Basis of Design Document (prepared by the SER) before detailed design can commence on the project; as such, the Peer Review Panel should be engaged as early as possible in the design process.

Reference Guidelines

The guidance provided in this section is different than that of the previous sections in that much of this practice involves important issues that are not addressed in the requirements of the current Code. As such, the guidance provided in this section is a consensus summary combining the research and expertise of Engineering Professionals who are subject matter experts on the life safety performance evaluation of tall concrete buildings in BC with the requirements of guidelines from other jurisdictions.

This section of the guidelines should be used in conjunction with one of the following two documents:

- *An Alternative Procedure for Seismic Analysis and Design of Tall Buildings in the Los Angeles Region*, published by the Los Angeles Tall Buildings Structural Design Council (referred to in this document as the "LATBSDC guidelines") (LATBSDC 2020); or
- *Guidelines for Performance-Based Seismic Design of Tall Buildings*, published by the Pacific Earthquake Engineering Center Tall Buildings Initiative (referred to in this document as the "PEER TBI guidelines") (PEER TBI 2017).

This section is not intended to be a comprehensive summary of all required procedures for the evaluation of life safety performance of tall concrete buildings using Non-linear Dynamic Analysis; where information is not presented here, the procedures described in either the LATBSDC guidelines or PEER TBI guidelines should be used. However, the SER is reminded that Canadian code requirements must always be met; in some instances, the guidance in the two reference guidelines does not meet the minimum requirements of Canadian codes and should not be applied to the design of tall concrete buildings in BC.

Where no specific clauses from Canadian codes or references to the LATBSDC guidelines or PEER TBI guidelines are mentioned in this section, the guidance provided is based on subject matter expert research and experience. In some cases, research is ongoing and no consensus could be reached at this time; in those cases, the necessary considerations for each procedure (based on the LATBSDC guidelines and PEER TBI guidelines) are provided, and the SER must use professional judgment to determine which are appropriate for a specific project.

The SER and the Peer Review panel for the project must ensure the procedure being followed is comprehensive and an appropriate combination of the procedures described in one or more of the reference guidelines and this document. The complete procedure that is selected must be described in the Basis of Design Document prepared by the SER and approved by the Peer Review panel.

Performance Objectives

The performance objective of the seismic design requirements in the Code is to:

- protect the life and safety of building occupants and the public as the building responds to ground shaking that has a probability of exceedance of 2% in 50 years (annual exceedance probability of 1/2,475).

Non-linear Dynamic Analysis is used to evaluate the life safety performance of a building. The life safety performance level corresponds to significant damage in the structure and loss of stiffness; however, at this performance level, the structure still has reserve capacity before reaching the collapse level.

The LATBSDC guidelines and PEER TBI guidelines include procedures for evaluating the "collapse prevention" performance of buildings subjected to risk-targeted maximum considered earthquake (MCE_R).

The current Code does not have an explicit performance objective requiring that buildings of normal importance (Article 4.1.2.1. of the Code) withstand more frequent and more moderate-intensity earthquake shaking with limited damage. However, for many buildings, particularly regular buildings, the prescriptive seismic design requirements of the Code will result in limited damage from lower levels of ground shaking that have a higher probability of occurrence.

The *NBC 2020* has a new requirement for limiting damage caused by ground shaking in normal importance buildings in seismic category SC4 with a height of more than 30 m, which specifies that:

- structural framing elements not considered part of the SFRS must be designed to remain elastic during 10%-in-50-year level ground shaking.

It is expected that design of buildings with irregular Gravity-Load Resisting Frames will be most influenced by this new requirement. While it is not a requirement of the current Code, the SER should consider meeting this additional performance objective when using Non-linear Dynamic Analysis to design a tall concrete building, particularly if the building has an unusual irregularity.

The Structural Commentaries to the *NBC 2020* states that where non-linear analysis is used to determine the seismic demands on a building, all the general and specific requirements of Subsection 4.1.8. of the *NBC 2020* still apply. Thus, a building designed using Non-linear Dynamic Analysis must meet the minimum strength requirements of the Code. Similarly, the guidance discussed elsewhere in Section 3.4 Design for Earthquake Ground Motions still applies, unless superseded by the requirements of this section.

In certain cases, Non-linear Dynamic Analysis will result in higher demands than Linear Analysis. Example 1: Two Towers on a Common Podium in Appendix B: Non-linear Dynamic Analysis Examples presents the results from Non-linear Dynamic Analysis on a building with two towers on a common podium, where the Non-linear Dynamic Analysis resulted in higher demands than Linear Analysis.

Simplified Procedures in CSA A23.3

The procedures used for Non-linear Dynamic Analysis have advanced considerably in recent years; however, like any engineering tool, they have limitations. One well-known limitation is the inability to account for all the complexities of shear demands on reinforced concrete. The fibre models used for concrete shear walls give an accurate estimate of the non-linear response of walls subjected to axial load and bending moment, but they do not account for all aspects of shear stresses and shear strains, some of which can be significant. For example, the plastic hinge region of a concrete wall with diagonal cracks will experience significant shear strains (interstorey drift ratios) in the absence of any applied shear force, but shear spring models are not able to account for that effect.

CSA A23.3 contains a number of simplified solutions for complex non-linear problems. For example, the simplified envelope of interstorey drift demands on Gravity-Load Resisting Frames (CSA A23.3, Clause 21.11) accounts for the interstorey drift ratio resulting from the shear strains in the plastic hinge regions of walls. Other examples include simple static procedure for determining backstay forces, simple estimates

of non-linear magnification of higher mode shear accounting for shear ductility, and procedures for ensuring the maximum compression strains in concrete walls are within acceptable limits.

In some cases below, options are given for either using the methods in CSA A23.3, or the procedures given in the LATBSDC guidelines or the PEER TBI guidelines. For example, the maximum strains in a concrete wall can be evaluated using the inelastic rotational capacity/ demand procedures in CSA A23.3, or they can be evaluated from strains directly. The latter approach requires a sensitivity analysis to confirm that a sufficient number of elements have been used in the model to make a good estimate of the maximum strain demand.

In other cases, the procedures given in CSA A23.3 are mandatory because the guidance provided in the two reference guidelines do not meet or exceed the requirements of the Code. An example is that the minimum interstorey drift ratio of a building over the height of the plastic hinge must be greater than the interstorey drift ratio resulting from the shear strains in the plastic hinge regions of walls.

3.4.8.2 Modelling Considerations

System Idealization

A three-dimensional model of the building should be used to represent the spatial distribution of (horizontal and vertical) mass, and the stiffness of the structure. As such, the model should incorporate realistic estimates of stiffness and damping considering the expected levels of excitation and damage.

The characteristics of the non-linear cyclic response of the structural elements in the model, such as stiffness, strength, ductility capacity, and hysteretic behaviour, must be representative of the behaviour of actual elements that have been subjected to reversed cyclic loading tests in the non-linear range.

The material properties to be used in the analysis are discussed further in Section 3.4.8.5 Evaluation of Life Safety Performance.

Gravity-Load Resisting Elements

The SFRS is designed to resist 100% of the seismic demands determined using Linear Analysis, as discussed above. The non-linear model needs to include the Gravity-Load Resisting Frame members to:

- accurately model the entire structure;
- capture any influence of the Gravity-Load Resisting Frame; and
- evaluate the life safety performance of this part of the structure.

The collapse of many concrete buildings in earthquakes originates within the Gravity-Load Resisting Frame. To accurately account for second-order (P-Delta) demands in a building, including the variation of lateral drift at different points on each storey, all gravity load columns and other structural elements in the Gravity-Load Resisting Frame need to be included in the three-dimensional model of the building. The stiffness of these members is expected to degrade more rapidly due to seismic demands, and this must be accounted for.

An ideal Gravity-Load Resisting Frame does not attract significant seismic loads. This would be the case, for example, when thin flat-plate slabs are supported on widely spaced flexible columns. When Gravity-Load Resisting Frames have significant lateral stiffness, the reduced period of the building increases the force demands on the SFRS.

A common way for the Gravity-Load Resisting Frame to resist lateral loads is by the floor slab (or beams) and columns acting as a moment-resisting frame. This phenomenon is commonly referred to as the “outrigger effect.” The lateral stiffness of the Gravity-Load Resisting Frame increases as the stiffness of the floor slab increases, or when the columns are more closely spaced.

Appendix C of the LATBSDC guidelines provides recommendations on how to model the coupling of the slab between the core and gravity-load columns, or between two gravity-load columns using outrigger beams. Following the LATBSDC guidelines, it is recommended that the outrigger effect be explicitly

modelled when a column is located within 6 m of the core, or when two columns are spaced at less than 3 m. These limits are applicable to thin flat-plate slabs, and proportionally larger spacing limits apply for thicker floor systems.

The Gravity-Load Resisting Frame must be designed to resist any forces that it attracts. As the failure mode of Gravity-Load Resisting Frame members may be very brittle, a safe estimate of the force must be made. Rather than using an upper-bound estimate of Gravity-Load Resisting Frame stiffness, which will reduce the displacement demands on the building, a factor of safety is applied to the estimate of the critical force demands. A good example of critical force demands is the axial compression that develops in the columns due to the outrigger effect.

Any non-structural components, such as architectural walls, that influence the displacements demands on the structure (e.g., reducing displacements on one side of the building and increasing displacements on the opposite side) must be accounted for.

Floor Diaphragms

When the concrete walls in a building are relatively uniform over many levels, the concrete slabs that interconnect these walls can be modelled as rigid, in-plane elements over those levels. Conversely, if there is a significant change or discontinuity in the walls or in any of the vertical elements in the Gravity-Load Resisting Frame, the flexibility of the diaphragms must be modelled in order to obtain a reasonable estimate of the force transferred by the diaphragms. The diaphragms must also be explicitly modelled if there are irregularities in the diaphragm, such as re-entrant corners or large openings, in order to determine the edge (chord) forces, as well as the shear, axial, and bending stresses in the diaphragms.

Diaphragms are normally designed to experience Elastic Deformations; hence, they can be modelled using elastic finite elements employing an effective stiffness that depends on the expected level of cracking of concrete. One challenge is that the range of possible effective stiffness is very large, and therefore it is difficult to make an accurate estimate of the effective stiffness until some estimate of the forces is known. An appropriate finite-element mesh is needed to correctly model the diaphragm flexibility.

Horizontal Mass

The horizontal seismic mass is determined based on the expected seismic weight of the building, including the dead load, and other loads as specified in Article 4.1.8.2. of the Code.

The mass must be accurately distributed in plan (i.e., the horizontal plane), in order to determine the torsional inertial effects.

While the mass of the ground floor should be included in the model, it is not simple to determine whether the mass of the below-grade structure should also be included. As the lateral stiffness of the below-grade soil is normally not accounted for, the mass of the full below-grade structure is also not normally included. The LATBSDC guidelines provide additional guidance on this matter.

Vertical Mass

For most buildings, the vertical mass does not need to be included in the analysis of the building subjected to horizontal ground accelerations.

Whenever the structural configuration is such that the horizontal motions in the building can induce vertical motions, the vertical mass must be included in the analysis. An example of where the vertical mass must be included is an inclined member, such as a sloped column, which will connect the horizontal and vertical modes of the building. See Section 3.4.7.2 Sloped-Column Irregularity for more information.

When the vertical mass is included in the analysis, the mass must be included in the model with sufficient accuracy in horizontal distribution to determine the numerous vertical modes of response correctly.

Vertical Ground Motions

For most buildings, the effect of vertical ground accelerations can be ignored, as the vertical-load-carrying members are designed for a minimum axial load equal to 1.4 times the dead load as part of the gravity-load design of the structure. The vertical ground motions need to be included in the analysis for any of the following cases, when:

- significant discontinuity exists in a vertical-load-carrying element, such as a gravity-load column supported on a transfer slab or girder;
- a GILD irregularity exists in the building according to the Code; or
- a vertical member, inclined more than 2 degrees from the vertical, supports a portion of the weight of the building in axial compression (Type 10 irregularity according to the *NBC 2020*).

Where vertical ground motions are included in the analysis, the vertical component of mass must be included in the model with sufficient accuracy in horizontal distribution to determine the numerous vertical modes of response correctly.

Damping

In non-linear analysis, the damping provided by hysteretic energy dissipation of structural members is modelled explicitly. An additional small amount of equivalent viscous damping may be included in the model to account for the inherent damping of the structure that is not associated with the response of non-linear elements. Damping is reduced in tall concrete buildings compared to low-rise buildings primarily because the relative damping contributions from foundations is smaller.

Following the LATBSDC guidelines, it is recommended that the effective additional modal or viscous damping for the primary modes of response must not exceed the following fraction of critical damping:

$$\zeta_{critcal} = \frac{0.20}{\sqrt{H}}$$

where H is the height of the roof in metres, excluding mechanical penthouses, measured from grade level.

The fraction of critical damping ($\zeta_{critcal}$) must not be taken more than 0.05, and need not be taken less than 0.025.

Viscous damping may be modelled using modal damping, Rayleigh damping, or a combination of the two. Use of modal damping for all modes may result in overestimation of floor acceleration, according to the LATBSDC guidelines. To alleviate this problem, the LATBSDC guidelines recommend using a combination of modal damping and stiffness-proportional damping, or linearly increasing damping from 0.2 T to the period of 0 seconds, such that the damping values are less than or equal to the specified critical damping for the entire period range of interest (0.2 T to 2 T).

P-Delta

Geometric non-linearity due to second-order bending moments (P-Delta) must be taken into account in the non-linear analysis using the appropriate gravity loads, in accordance with Article 4.1.3.2. of the Code, regardless of whether any elastic analysis indicates that such effects are important. The total gravity loads for the entire building must be included, and these loads must be accurately distributed to capture the influence of both building translation and twist.

Gravity Load

Superposition of design forces cannot be used with non-linear analysis results. Therefore, the gravity loads that occur simultaneously with the earthquake forces must be applied to the model of the building for the non-linear earthquake analysis.

The gravity loads that need to be considered in the non-linear model must be consistent with the Code load combinations. According to the Code, the companion loads, 50% live load (0.5L) and 25% snow load (0.25S),

are only applied if they have a detrimental effect. Thus, the two load combinations to be considered are:

$1.0D + 1.0E + 0.5L + 0.25S$; and

$1.0D + 1.0E$.

Normally, it is not necessary to repeat the non-linear analysis with the second load combination; however, consideration should be given to the effect of the reduced gravity loads on the structure. The PEER TBI guidelines provide guidance on when the second load combination should be considered. The Basis of Design Document prepared by the SER and approved by the Peer Review panel must address this issue.

Torsion

The distribution of horizontal mass in the model must reflect the actual conditions in the building, so that any inherent torsional eccentricity will be accounted for in the analysis.

When accidental eccentricity (causing torsion) must be added to the structure, this should be done by displacing the centre of mass by 5% of the building dimension perpendicular to the direction under consideration. Where earthquake forces are applied concurrently in two orthogonal directions, the required 5% displacement of the centre of mass need not be applied in both of the orthogonal directions at the same time, but should be applied in the direction that produces the greater effect.

Shifting the centre of mass in four different directions significantly increases the number of non-linear analyses that need to be done. In order to reduce the number of non-linear analyses, both the LATBSDC guidelines and the PEER TBI guidelines suggest that accidental eccentricity need not be added to the non-linear model when a building has low torsional sensitivity.

The LATBSDC guidelines recommend using the results of Linear Dynamic Analysis with only the inherent torsional eccentricity (no accidental eccentricity), to calculate the twisting index:

$$A = \left(\frac{\delta_{max,t}}{1.2\delta_{avg,t}}\right)^2$$

If A exceeds 1.5 for any floor where the maximum average drift ratio exceeds 0.010, then accidental eccentricity must be considered during non-linear analysis.

The PEER TBI guidelines approach requires two separate analyses using either the modal response spectrum method or the equivalent lateral force procedure. The floor level displacements are calculated considering only inherent torsional eccentricity in the first, while accidental eccentricity (causing torsion) is also applied in the second. The accidental torsion is generated by displacing the centre of mass each way from its actual location by a distance equal to 5% of the dimension of the structure perpendicular to the direction of the applied forces. The results of the two analyses are used to calculate the twisting index:

$$A = \left(\frac{\delta_{max,ta}}{\delta_{avg,t}}\right)$$

If $A > 1.2$ at any level, accidental eccentricity must be included in non-linear analysis.

A recommendation for the approach to be used in all cases to account for accidental eccentricity (causing torsion) is not provided at this time. The SER must determine which approach best applies to the project, provide justification in the Basis of Design Document, and have it approved by the Peer Review panel before commencing design.

Backstay Effect

When the plastic hinge in the core occurs immediately above grade level, the multiple static analysis procedure described in Section 3.4.5 Refined Analysis of Structure Below Plastic Hinge Zone can be used to determine the design forces in the structure below the plastic hinge, including the foundation forces. This avoids having to do multiple Non-linear Dynamic Analyses with varying effective stiffnesses for the elements below the plastic hinge.

When the plastic hinge occurs above podium levels that include significant mass contributing to the seismic demands below the plastic hinge, the calculation of design forces below the plastic hinge should be determined using multiple Non-linear Dynamic Analyses.

The structure below the plastic hinge is typically an indeterminate system with multiple load paths for the lateral loads. The shear and bending moment in the core can be transmitted directly to the foundation, or can be transferred to other walls by the diaphragms that interconnect the walls. The results of the analyses are very sensitive to the assumed in-plane stiffness of the diaphragms. If the forces in the diaphragms are low, the stiffness will be very high; but as the diaphragms crack due to applied forces, the stiffnesses reduce rapidly. As a result, multiple analyses must be done to bound the solution.

Appendix A of PEER/ATC-72-1, Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings (PEER and ATC 2010) provides further discussion and guidance on design and modelling considerations to address the backstay effect.

Foundation Modelling

The LATBSDC guidelines present a summary of different approaches that can be used to model the subterranean levels, including the foundation.

The recommended approach is to model the concrete structure down to the foundation level while excluding the surrounding soil. The subterranean structure is typically a stiff indeterminate system of walls consisting of the core walls, the perimeter foundation walls, and other columns and walls, which are inter-connected by the floor diaphragms. Appropriate assumptions need to be made for effective stiffnesses of the subterranean elements, particularly the diaphragms.

The Code requires that the increased displacements of the structure resulting from foundation movement (soil flexibility) be accounted for. This effect can be easily incorporated into the static analyses done in accordance with Section 3.4.5 Refined Analysis of Structure Below Plastic Hinge Zone, and does not need to be included in the Non-linear Dynamic Analysis if the CSA A23.3 static analysis approach is used to determine the increased displacements of the structure resulting from foundation movements and cracking of the subterranean structure.

The earthquake ground motions should be applied at the base of the subterranean structure and can be either free-field motions or modified motions due to kinematic interaction effects. The optional soil-structure interaction can typically be ignored in the non-linear analysis of tall concrete buildings. The National Institute of Standards and Technology (NIST) provides guidelines about when soil-structure interaction is likely to significantly affect the fundamental-mode response of buildings depending on the structure height, fixed-base building period, and soil shear wave velocity. This condition rarely applies to tall concrete buildings.

Modelling of Structural Components

Component monotonic backbone curves and cyclic deterioration characteristics must be established from physical test data, or from analytical approaches that have been benchmarked to physical test data.

Sources of deterioration in concrete structures must be accounted for unless precluded by detailing and/or capacity design, including:

- concrete cracking, crushing, and spalling;
- reinforcement yielding, buckling, and fracture;
- reinforcement bond slip and anchorage failure;
- shear friction sliding and failure;
- concrete dilation; and
- confinement steel yielding and failure.

Analysis models for overall structural system response range from concentrated hinge or spring models, to fibre elements, to detailed continuum finite-element models. All models must be calibrated to physical test data at either the material, subcomponent, or component level.

The valid range of deformation capacities for components should be established from analytical models validated by physical test data, directly from physical test data, or taken from ASCE 41, Seismic Evaluation and Retrofit of Existing Buildings. The ASCE 41 component-force-versus-deformation models can be used as first-cycle envelope curves; however, in reality, the decrease of resistance beyond the point of peak strength indicated by ASCE 41 is not likely as rapid unless fracture occurs. This rapid decrease of resistance may cause numerical instabilities in the analysis process. Alternatively, the modelling options presented in the *Guidelines for Nonlinear Structural Analysis for Design of Buildings, Part I – General* (NIST 2017) and PEER/ATC-72-1 may be used.

The component models must account for post-peak strength and stiffness deterioration due to cyclic loading, or the ultimate deformation of the component must be limited to the point at which the model fails to accurately represent the response. The recommended approach is to explicitly model the strength and stiffness deterioration that occurs under cyclic loading using an algorithm that adjusts the response from the monotonic response to some deteriorated response that is a function of cyclic loading.

Concrete Walls

The response of concrete walls subjected to axial load and bending moment is modelled using plane sections analysis. The vertical normal strains in the walls are assumed to vary linearly, and stress-strain relationships for the concrete and reinforcement account for the influence of cyclic degradation.

Typical concrete stress-strain curves are given by Collins and Mitchell (1997) and suitable adjustments to account for confinement are described by Mander et al. (1988) and Saatcioglu and Razvi (1992). High-strength concrete is considerably more linear and more brittle than normal strength concrete; this effect must be accounted for when selecting a stress-strain curve to be used in the fibre model.

The stress-strain responses of reinforcement under cyclic load are highly non-linear due to the Bauschinger effect. It is difficult to include deterioration due to reinforcement-localized buckling and fracture in the steel stress-strain curve, and thus the vertical strain in the wall must be limited to account for these critical deterioration modes.

The height of the fibre element in the wall must be limited to ensure a reasonable estimate of maximum inelastic strains in the plastic hinge region of the wall. The inelastic strains in a wall vary approximately linearly in the plastic hinge region, but it is common to assume the maximum inelastic strains are uniform over an idealized plastic hinge height (that is half the height that the inelastic strains actually vary linearly). The two different variations of inelastic curvatures give similar top wall displacements. The height over which the inelastic strains vary linearly defines where the special detailing is required in the wall. CSA A23.3 defines this height as $0.5l_w + 0.1h_w$, where l_w is the length of the wall and h_w is the height of the wall. A reasonable estimate of the height over which the inelastic strains can be assumed to be approximately uniform is $0.2l_w + 0.05h_w$. The height of the elements should not exceed this height or the storey height. For tall concrete buildings, the storey height usually governs.

When compression strains in the wall are large, and therefore critical, it may be necessary to use multiple elements over the length of the wall in order to make a good estimate of the maximum compression strain at the ends of the wall.

A fibre model should be used over the full height of the building in order to accurately estimate the higher mode shear demands. In the early practices of Non-linear Dynamic Analysis of tall concrete buildings, it was common to model the wall outside the plastic hinge region using elastic elements. However, this practice cannot accurately capture the flexural cracking and small amount of vertical reinforcement yielding near mid-height of the wall present in the higher mode analysis, and may therefore overestimate the higher mode shear demands.

Coupling Beams

Coupling beams in BC typically contain diagonal reinforcement.

The strength of the coupling beams controls the capacity of coupled wall systems. Any overstrength in the coupling beams that is not correctly accounted for will result in larger tension and shear demands on the wall piers.

As a coupling beam is subjected to reverse cyclic demands, the beam tends to "grow" in length and the concrete slab that surrounds the coupling beam will restrain this "growth." The resulting axial compression applied to the coupling beam will increase the flexural (and shear) capacity when it is diagonally reinforced. Longitudinal reinforcement in the slab adjacent to the coupling beam will provide additional overstrength.

Caution is needed when fitting a backbone curve to test results, to ensure the overstrength is correctly included.

Transfer Slabs

Appendix C in the LATBSDC guidelines provides guidance on the modelling of slab-column frames using linear elastic outrigger beams. The methodology was developed for cases where the slabs are relatively thin.

Thick transfer slabs are sometimes used in tall concrete buildings in BC. These members can have a very pronounced influence on the demands on the gravity-load columns, and even influence the demands on the core. Thus, special attention needs to be paid to the accurate modelling of these elements.

Slab–Column Connections

Slab–column connections can be represented using either an effective beam-width model or an equivalent-frame model. Where deformations exceed the yield point at a connection, it may be convenient to insert a non-linear rotational spring between the components representing the slab and the column. Further information is given in PEER/ATC-72-1 and ASCE 41.

3.4.8.3 Required Number of Analyses and Assumed Component Strengths

Types of Demands (Actions)

Seismic demands (actions) can be classified as either Deformation-Controlled Demands or Force-Controlled Demands.

Deformation-Controlled Demands include deformations and forces associated with a ductile non-linear response under reversed cyclic loading. This includes:

- inelastic rotation of concrete walls;
- inelastic rotation of coupling beams; and
- deformation demands on ductile elements of the Gravity-Load Resisting Frame.

Deformation-Controlled Actions are permitted in SFRS elements that are specifically designed and detailed in accordance with CSA A23.3 to exhibit a ductile non-linear response under reversed cyclic loading. CSA A23.3, Clause 21.11 also specifies the detailing required in Gravity-Load Resisting Frame members to tolerate the imposed deformation demands.

Force-Controlled Demands in a highrise core wall building include:

- shear force demand on the walls;
- forces in diaphragms at podium levels and other levels of discontinuity in the SFRS;
- overturning moment applied to the foundation; and
- forces applied to the Gravity-Load Resisting Frame members, except bending moments when the member is modelled as a non-linear element.

NBC Structural Commentaries

The Structural Commentaries to the *NBC* 2015 states that two full sets of non-linear analyses need to be done with different component strengths to determine separately the Deformation-Controlled Demands and the Force-Controlled Demands. As per the *NBC 2015*, lower-bound component strengths need to be used when determining the Deformation-Controlled Demands, while upper-bound component strengths need to be used to determine the Force-Controlled Actions.

The lower-bound strength recommended by the *NBC 2015* is 1.1 times the nominal strength, which corresponds to component strengths calculated using concrete and reinforcement material strengths of $1.1f_c'$ and $1.1f_y$, respectively. A lower-bound estimate for the increase in reinforcement stress due to strain hardening should be included in the model. These lower-bound strength values result in upper-bound estimates of Deformation-Controlled Demands on the SFRS.

The upper-bound strengths recommended by the *NBC 2015* to determine Force-Controlled Demands are 1.2 times the probable resistance, which corresponds to $1.2f_c'$ and $1.5f_y$ (1.2 x $1.25f_y$). An upper-bound model for the increase in reinforcement stress due to strain hardening must also be included in the model.

Highrise Buildings in BC

The LATBSDC guidelines and PEER TBI guidelines, which have been used for the design of many core wall buildings, require a minimum of one set of non-linear analyses with one set of 11 ground motions, and use expected component strengths calculated using concrete and reinforcement material strengths of $1.3f_c'$ and $1.17f_y$, respectively.

The unique seismicity in BC requires that at least two sets of 11 ground motions be used for the non-linear analysis; see subsection Number of Ground Motions under Section 3.4.8.4 Seismic Hazard for more information. These analyses need to be repeated four times if accidental torsion is included, and even more times if the backstay forces are to be calculated using non-linear analysis.

It is recommended that one set of component strengths be used, calculated using concrete and reinforcement material strengths of $1.2f_c'$ and $1.2f_y$, respectively.

The recommendation for the component strengths mentioned above is based on the following. The typical grade of reinforcement in BC is 400W, meeting CSA G30.18, Carbon Steel Bars for Concrete Reinforcement, which has a minimum yield strength of 400 MPa and a maximum yield strength of 525 MPa. Suppliers typically target mid-way between these limits, i.e., 460 MPa. The minimum ultimate strength according to CSA G30.18 is the larger of 540 MPa and 1.15 times the actual yield strength. Thus, reinforcement typically used in BC will often have an actual yield strength of about 460 MPa and minimum ultimate strength of 540 MPa.

3.4.8.4 Seismic Hazard

This section provides guidance for the selection and scaling of time histories to be used in the Non-linear Dynamic Analysis of tall concrete buildings. It generally follows the procedures given in Commentary J of the *NBC 2015*; with modifications based on the consensus of recommended practice in BC.

As the determination of seismic hazard values and the Non-linear Dynamic Analysis of tall concrete buildings are very specialized areas of practice, the SER and geotechnical engineer of record (GER) should work together to determine the division of responsibilities amongst themselves and/or specialists under their direct supervision.

The GER is responsible for determining the required soil properties and their uncertainties, and the responsibility of the following items may vary by project and expertise of the Registered Professionals of Record:

- Select a suitable suite of input ground motions
- Perform site-specific response analysis (SSRA)
- Where required, interpret the results in a manner conducive for use with probabilistic seismic hazard analysis (PSHA)

Regardless of who is responsible, the SER should understand the principles of the site-specific investigation and include the results of the site-specific investigation and analysis in the Basis of Design Document, and should also participate in any conversations required between the Registered Professionals of Record and the Peer Review panel, to agree on the appropriate approach for the project.

Design Spectrum

The design spectrum used for ground-motion selection is as specified by the Code for Linear Dynamic Analysis, except that at periods less than 0.5 s, no "cut-off" (plateau) is required. To determine spectral acceleration values at other period values, "Log (T) – Log (S)" interpolation should be used, as linear interpolation of acceleration at widely spaced periods (2 s, 5 s, or 10 s) results in a highly distorted displacement spectrum.

Site-specific PSHA can be performed to develop an alternative spectrum. *The Sixth Generation Seismic Hazard Model of Canada* (Kolaj et. al, 2020), which was developed for the seismic design values in the *NBC 2020*, should be used to determine seismic hazard at the site.

For a very tall concrete building with an unusual irregularity in the Lower Mainland of BC, a site-specific PSHA should be performed using *The Sixth Generation Seismic Hazard Model of Canada* plus the explicit consideration of basin effect.

Canada's sixth generation seismic hazard model retains most of the seismic source model from the fifth generation (used for seismic data in the *NBC 2015* and the *BCBC 2018*); but updates the earthquake sources for the deep in-slab earthquakes under the Strait of Georgia and adds the Leech River–Devil's Mountain fault near Victoria, BC. The rate of Cascadia earthquakes of approximately magnitude9 is also increased to match new paleoseismic information. Two major changes are updating ground motion models, and the use and adaptation of various ground motion models to directly calculate hazard on various site classes with representative V_{s30} values, rather than providing hazard values on a reference Class C site and applying *F*(*T*) factors, as in the fifth-generation hazard mode used in the *NBC 2015*. The sixth-generation seismic hazard model accounts for basin effects in an implicit way.

Shear Wave Velocity

For a discussion on how to determine V_{s30}, see the subsection Site Class in Section 3.4.3.2 Seismic Demands.

Site-Specific Response Analysis

SSRA, which is also called site-specific ground response analysis, can be used to determine the spectrum for a site. The resulting spectrum cannot fall below 80% of the Code uniform hazard spectrum.

SSRA involves analysis of wave propagation through the soil medium to assess the effect of local geology on the ground motion. Thus, SSRA can be more accurate than the Code uniform hazard spectrum modified by a site class when done correctly; however, the procedure to arrive at the spectrum includes assumptions and judgment that may strongly influence the result. Thus, the Peer Review panel must review the SSRA results to ensure the analysis was done correctly. See Section 4.3 Peer Review for more information.

A reduced spectrum can be described as reduced demands on the building due to "yielding" of the soil below the building. Any higher-than-expected soil strength (overstrength) will result in higher demands on the building. Thus, upper-bound estimates of soil strengths must be considered in the analysis.

SSRA does not provide a valid representation of site effects for periods beyond the elongated fundamental period of the soil column used in the analysis. Special procedures are required for merging the results of SSRA at short periods with ergodic models at long periods to reduce the potential for bias (Stewart et al. 2014).

Period Range

A period range (T_R) with a lower-bound (T_{min}) and an upper-bound (T_{max}) must be defined such that it includes the periods of the structure's significant modes of vibration (Figure 5 below).

- T_{min} is taken as the smaller of $0.15T_1$ or $T_{90\%}$, where:
  - T_1 is the fundamental period of the structure based on the effective stiffness of concrete walls given in CSA A23.3 (as a function of elastic bending moment to strength of the wall); and
  - $T_{90\%}$ is the lowest period of the modes necessary to achieve 90% mass participation.
- T_{max} is taken as the larger of $2.0T_1$ and 1.5 s.

The upper-bound range of period may be reduced to $1.5T_1$, if it can be shown that the average period elongation obtained from the analysis using the suite of ground motions does not exceed $1.5T_1$. This requirement is another example of guidance that is based on consensus of subject matter experts, considering requirements of the Code and guidance provided in the LATBSDC guidelines and the PEER TBI guidelines.

If the non-linear analysis results are not used for design of subterranean elements, $T_{90\%}$ can include only the mass of superstructure.

Where vertical response is considered (i.e., where vertical mass must be included, as described in Section 3.4.8.2 Modelling Considerations), the lower-bound of the period range may not be taken less than the larger of 0.1 s or $T_{90\%}$ in the vertical direction.

Appendix X of Commentary J of the *NBC 2015* recommends a minimum of one scenario-specific period range (T_{RS}) for each tectonic environment (or source) contributing to the hazard including crustal, in-slab, and subduction interface earthquakes (i.e., three T_{RS} comprised of one each: $T_{RS\ Crustal}$, $T_{RS\ Subcrustal}$, and $T_{RS\ Interface}$) in southwestern BC. Based on the consensus of subject matter experts consulted in the development of these guidelines, it is acceptable to combine crustal and in-slab sources and have one specific period range ($T_{RS\ short}$) over the short period range according to disaggregation results. The second specific period range ($T_{RS\ long}$) must be developed over the long period range for subduction interface motions.

The site-specific PSHA disaggregation should be performed to select motions that are compatible with the tectonic regime (e.g., active crustal regions, subduction zones) and controlling distance, magnitude, and site condition.

Method A and B of Appendix X of Commentary J of the *NBC 2015* can be used to develop the target response spectra ($S_T(T)$) for the horizontal component of ground motions.

Disaggregation results should be used to determine the dominant tectonic regime, magnitude, and distance contributing over the period range (T_R) and divide the period range into scenario-specific period range (T_{RS}).

Recent hazard models developed for the *NBC 2020* provide data only up to a period of 10 s; therefore, caution should be used when the fundamental lateral period of the building (T_1) is greater than 5 s.

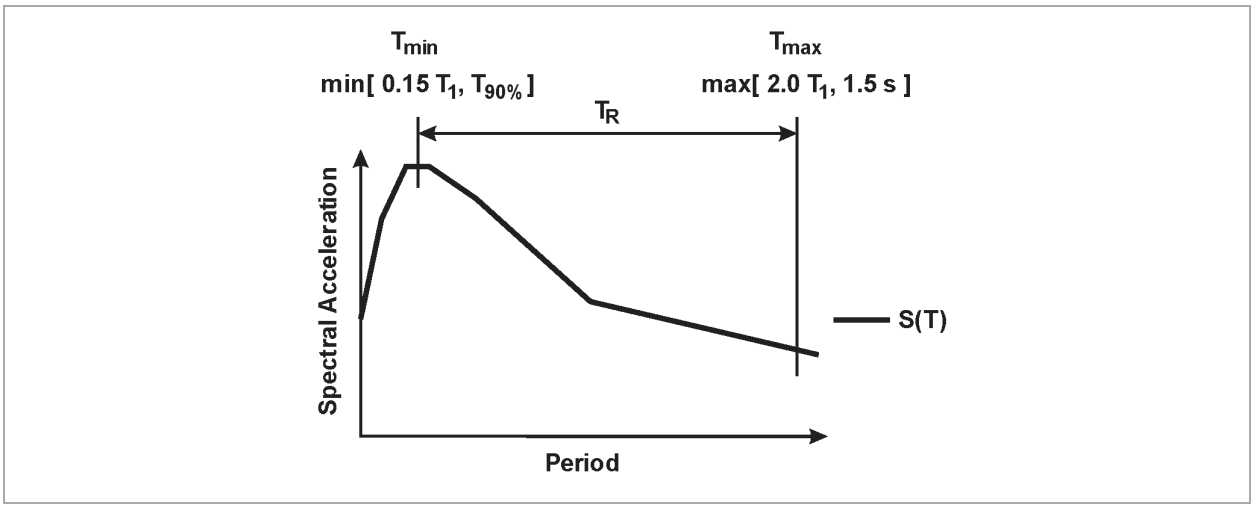


Figure 5: Period range used for selection and scaling of ground motions

Number of Ground Motions

A minimum of two sets of 11 ground motions should be used (a minimum of 22 ground motions in total). One set of motions are matched to the target spectrum over $T_{RS\ short}$ (combined crustal and in-slab sources period range), and the other set is matched over $T_{RS\ long}$ (interface subduction source period range).

While the Structural Commentaries to the *NBC 2015* recommends not less than 11 ground motions over three scenario period ranges (33 motions in total), it permits as few as three sets of 5 ground motions (15 motions in total). Based on the consensus of subject matter experts, 11 ground motions over two scenarios (22 ground motions) is the recommended practice in BC.

No more than two ground motion records should be selected from the same earthquake event for short period (crustal and in-slab) sources. Due to scarcity of recordings of large magnitude interface subduction events, it is permitted to use up to 4 (out of 11) ground motions from the same interface subduction event.

The selected ground motions should reasonably reflect the anticipated duration of the design earthquake. For this reason, it is recommended to ensure the mean significant duration (such as D_{5-95}) of selected interface ground motion records reflects the long duration effect of subduction interface scenarios in BC.

Depending on the disaggregation results, it may be appropriate to include near-fault records to account for rupture directivity and fling-step effect.

Scaling of Ground Motions

Pairs of horizontal ground motions components should be scaled with a single factor so that the geometric mean of the spectra of the two horizontal components matches the target spectrum. The recommended approach based on subject matter expert consensus summary is that scale factors less than 0.5 or greater than 4 should not be used.

When linear scaling technique is used, the average of the geometric mean spectra from all ground motions must not fall below 90% of the target spectrum at every period of T_{RS}. When the spectral matching technique is used, the average of the geometric mean spectra from all ground motions should not fall below 110% of the target spectrum at every period of T_{RS}.

The vertical component of ground motions should be scaled by the same factor as the corresponding horizontal ground motion components. Consequently, compatibility of the scaled vertical component spectra with the target vertical spectra must be checked. Alternative methods may be considered where significant incompatibility is observed. Vertical target spectrum may be developed using relationships between vertical and horizontal spectra that depend on site and soil conditions (Stewart et al. 2016; Bozorgnia et al. 2010; Gülerce and Abrahamson 2011).

3.4.8.5 Evaluation of Life Safety Performance

Evaluation Criteria

To ensure the building meets the life safety performance level, the calculated response must satisfy all of the following requirements:

- Deformation demands on Deformation-Controlled Actions or elements are within the limits specified in CSA A23.3.
- Strength demands on Force-Controlled Actions or elements are smaller than the factored strengths calculated in accordance with CSA A23.3.
- A maximum of one unacceptable response occurs for the suite of 11 ground motions.
- Peak transient drifts and residual drifts are within acceptable levels.

The demand on the structure is determined from non-linear analysis using a model of the building having component strengths calculated using the “expected” concrete and reinforcement material strengths of $1.2f_c'$ and $1.2f_y$, respectively.

The deformation capacities and strength capacities are calculated using the procedures in CSA A23.3, with factored material strengths calculated from the specified material strengths, f_c' and f_y, times the resistance factor, 0.65 for concrete and 0.85 for reinforcement.

Design Seismic Demand Parameter

Two suites of 11 ground motions selected and scaled over scenario-specific period ranges should be used to conduct the non-linear analyses.

When there is no unacceptable response for the suite of 11 ground motions, the mean of the maximum values for each ground motion is determined.

When there is one unacceptable response for the suite of 11 ground motions, the demand is determined as 120% of the median value from the complete suite including the unacceptable case, but not less than the mean of the values for the 10 ground motions producing acceptable responses.

Finally, the design seismic demand parameter is equal to the larger demand determined from the two suites of ground motions.

Unacceptable Response

If spectral matching is not used to scale the two suites of 11 ground motion records, one unacceptable response is permitted. Otherwise, no unacceptable response is permitted.

Examples of unacceptable responses include:

- analysis stops due to convergence issues;
- demand on deformation-controlled element exceeds the valid range of modelling;
- demand on force-controlled element exceeds the element capacity;
- peak transient storey drift exceeds 4%; and
- residual storey drift exceeds 1.5%.

Global Response

To avoid unacceptable responses, both peak transient storey drift and residual storey drift must be considered when determining the global response of tall concrete buildings.

Peak Transient Storey Drift

The mean interstorey drift ratio demands must not exceed the regular limit of 2.5%, as specified in the Code. In addition, the maximum interstorey drift ratio from any one record must not exceed 4%.

Residual Storey Drift

The mean of the absolute values of residual drift ratios must not exceed 1%. In addition, the maximum residual storey drift ratio in any one analysis must not exceed 1.5%, unless approved by the Peer Review panel. See Section 4.3 Peer Review for more information.

Limiting the residual storey drift will protect against excessive post-earthquake deformations that likely will cause the building to be unrepairable. Large residual drifts are of particular concern for tall concrete buildings because of the danger a leaning building poses to the surrounding community. However, residual drift is not a life safety performance issue, and residual drifts are difficult to predict.

Nevertheless, residual drift in a building that has a GILD irregularity must be evaluated. During an earthquake, the lateral displacements of the building "ratchet" in the direction of the GILD. See Example 2: Gravity-Induced Lateral Demand (GILD) in Appendix B: Non-linear Dynamic Analysis Examples.

Evaluation of Core Walls

Core walls are designed to dissipate energy by flexural yielding of the wall piers and flexural/shear yielding of the coupling beams. The deformation demands on the wall piers are evaluated in the first section below, while the deformation demands on coupling beams are evaluated in the second. Finally, the shear force demands on ductile wall piers are evaluated in the third section.

Deformation Demands on SFRS: Wall Piers Within Plastic Hinge Region

Core walls are usually designed for flexural yielding to occur within the "plastic hinge regions" defined in CSA A23.3-14, Clause 21.5.2.1. This region contains special detailing, as described in Section 3.4.4.7 Seismic Detailing Requirements. As a result, this region of the wall is able to tolerate larger strain demands than the region of the wall outside the plastic hinge region; this is discussed separately below.

There are two procedures to evaluate the deformation demands on wall piers. The first is in terms of inelastic rotational demands and capacities and the second is in terms of maximum strains.

- **Procedure 1:** The Inelastic Deformation demands on wall piers are investigated in terms of wall rotations, which are the result of the inelastic curvatures over a height of wall equal to about half the height of the plastic hinge region, where special seismic detailing is provided. The inelastic rotational demands must be less than the (factored) inelastic rotational capacity.

  The inelastic rotational demand on the wall piers can be determined from the procedures in CSA A23.3, Clause 21.5.7.2 using the maximum top wall displacement determined from non-linear analysis, or it can be determined directly from the non-linear analysis as the maximum slope change over the plastic hinge region of the wall.

  The inelastic rotational capacity of the wall is calculated according to CSA A23.3, Clause 21.5.7.3, where the compression strain depth of the wall is calculated using the factored compression strength of concrete per CSA A23.3, Clause 21.5.7.4. Both the maximum compression strain and maximum tension strain limit the inelastic rotational capacity of the wall.

- **Procedure 2:** The Inelastic Deformation demands on wall piers are investigated directly in terms of compression and tension strains. The estimated maximum strains in a wall are very sensitive to the modelling assumptions, such as the height of the elements used to discretize the wall. Thus, a sensitivity analysis is required to confirm that a sufficient number of elements have been used to make a good estimate of the maximum strain demand. Analysis has shown that up to four elements per storey may be needed to make a good estimate of the maximum compression strain (LATBSDC guidelines).

  The maximum compression strains determined directly from the non-linear analysis must be increased by a factor of 2.0 to account for the high concrete compression strength ($1.2f_c'$) that is used in the non-linear analysis compared to the factored strength of concrete ($0.65f_c'$) to be used with the compression strain limits given in CSA A23.3. The maximum compression strain of concrete (2.0 times the value determined from non-linear analysis) must be limited to 0.0035 unless the compression region of the wall contains confinement reinforcement, and then the maximum compression strain must be limited as a function of the amount of confinement reinforcement per CSA A23.3, Clause 21.5.7.5.

  The maximum reinforcement tension strain must be limited to 0.05 to avoid fracture of the reinforcement accounting for tension stiffening, which causes a localization of the strains at the crack.

Outside Plastic Hinge Region

These regions of the wall require less special seismic detailing; lower strain limits are appropriate. The maximum compression strains in concrete (after multiplying by the factor of 2.0 described above) should be limited to 0.002 if the reinforcement at the end of the wall is tied as a compression member in accordance with CSA A23.3, Clause 7.6.5, and limited to 0.003 if the reinforcement at the end of the wall has buckling-prevention ties as per CSA A23.3, Clause 21.2.8.1.

The maximum tension strains in the reinforcement should be limited to 0.01.

When non-linear analysis indicates yielding of the wall outside the plastic hinge region due to higher-mode bending moments, the inelastic curvatures (and strains) are usually relatively small. If the analysis indicates significant yielding in the upper regions of the wall due to an irregularity in the building, such as a cut-off wall, or a transfer member framing into the core, the region should be designated as an additional plastic hinge region consistent with CSA A23.3, Clause 21.5.2.1.4, and the procedures described above for the plastic hinge region used to evaluate the deformation

demands. One of the advantages of a non-linear analysis is that it will reveal the effect of such irregularities (see Example 3: Discontinuous Shear Wall in Appendix B: Non-linear Dynamic Analysis Examples).

Deformation Demands on SFRS: Coupling Beams

The deformation demands on coupling beams must be limited using either one of two procedures. The first considers only the inelastic portion of the coupling beam rotations consistent with the procedures in CSA A23.3, while the second considers the total rotational demands on coupling beams.

- **Procedure 1:** The inelastic rotational demands on the coupling beams must be limited to the values given in CSA A23.3, Clause 21.5.8.4.5, as follows:
  - 0.04 for coupling beams with diagonal reinforcement meeting all requirements of CSA A23.3, Clause 21.5.8.2; or
  - 0.02 for coupling beams without diagonal reinforcement meeting all requirements of CSA A23.3, Clause 21.5.8.1.

  The inelastic rotational demands on the coupling beams can be determined from the top wall displacements using the procedures in CSA A23.3, Clause 21.5.8.4.4, or they can be taken directly from the non-linear analysis.
- **Procedure 2:** The total rotational demands on coupling beams must be limited as follows:
  - 0.06 for coupling beams with diagonal reinforcement meeting all requirements of CSA A23.3, Clause 21.5.8.2; or
  - 0.03 for coupling beams without diagonal reinforcement meeting all requirements of CSA A23.3, Clause 21.5.8.1.

Shear Force Demands on Wall Piers

The mean shear force demand determined from non-linear analysis must be less than the factored shear resistance calculated using the procedures in CSA A23.3, Clause 21.5.9, with the regular resistance factors from CSA A23.3 applied to the specified material strengths.

If the maximum shear force demand from a ground motion is greater than the nominal resistance calculated using resistance factors equal to 1.0 applied to the specified material strengths, it is considered an unacceptable response. See subsection Evaluation Criteria in Section 3.4.8.5 Evaluation of Life Safety Performance for more information on unacceptable responses.

The mean shear force demand determined from Non-linear Dynamic Analysis does not need to be increased (by the 1.3 or 1.5 factor used by the LABSDC guidelines and the PEER TBI guidelines, respectively) because the shear resistance determined from CSA A23.3, Clause 21.5.9 includes a safe limit on the diagonal compression stresses in concrete shear walls to avoid brittle compression-shear failure.

As described in Section 3.4.4.5 Design of Walls for Shear, within the plastic hinge region, the shear resistance is reduced as a function of the inelastic rotational demands calculated as part of the assessment of deformation demands on wall piers.

Example 5: Inelastic Effects of Higher Mode Shears in Appendix B: Non-Linear Dynamic Analysis Examples compares the shear force demands from Non-linear Dynamic Analysis with the design shear force specified by CSA A23.3 for two buildings. Non-linear Dynamic Analysis generally gives an upper-bound estimate of the shear force demands compared with the procedures in CSA A23.3 for estimating the same.

Force Demands on Other Members

All elements of the structure, including the Gravity-Load Resisting Frame members, must be checked for actions resulting from the combined gravity load and the demands from earthquake ground motions.

Gravity-load resisting elements can be included in the model of the structure, or they can be checked independently based on the results (deformations) determined from the Non-linear Dynamic Analysis. The gravity-load members can be modelled as linear-elastic elements allowing the Force-Controlled Demands to be assessed or, where appropriate, modelled as non-linear

elements allowing the deformation demands (bending moment) and Force-Controlled Demands (shear and axial force) to be assessed.

Critical Force-Controlled Actions

Member actions are classified as either Deformation-Controlled Actions or Force-Controlled Actions. Force-Controlled Actions are further classified into different categories of criticality. The Basis of Design Document prepared by the SER and approved by the Peer Review panel must identify the critical Force-Controlled Actions for the building.

Examples of what is usually considered a critical Force-Controlled Action in a tall core wall building include:

- shear demands on gravity-load columns;
- axial load demands on gravity-load columns acting as (unintentional or intentional) outriggers;
- shear and bending moment demands on transfer slabs and girders;
- in-plane shear demand on transfer diaphragms;
- force transfer between diaphragms and vertical elements of the SFRS; and
- shear force demands on foundation elements.

For critical force-controlled elements, the mean force demand determined from non-linear analysis must be less than the factored resistance calculated using the regular resistance factors from CSA A23.3 applied to the specified material strengths. When the maximum demand from a ground motion is greater than the nominal resistance, calculated using resistance factors equal to 1.0 applied to the specified material strengths, it is considered an unacceptable response. See subsection Unacceptable Response in Section 3.4.8.5 Evaluation of Life Safety Performance for more information on unacceptable responses.

For Force-Controlled Demands that are not considered critical, it may be appropriate to compare the mean force demand with a calculated resistance larger than the factored resistance calculated using the regular resistance factors from CSA A23.3 applied to the specified material strengths. This issue should be addressed in the Basis of Design Document prepared by the SER and approved by the Peer Review panel.

Slab-Column Connections

The demands on slab–column connections can be treated as a Deformation-Controlled Action, and the requirement for shear reinforcement can be determined in accordance with CSA A23.3, Clause 21.11.4, with the interstorey drift ratio determined from Non-linear Dynamic Analysis.

Interstorey Drift Ratio Due to Shear Strain

The fibre models used for concrete shear walls give an accurate estimate of the non-linear response of walls subjected to axial load and bending moment, but do not account for all aspects of shear stresses and shear strains. The plastic hinge region of a concrete wall with diagonal cracks will experience significant shear strains (interstorey drift ratios) in the absence of any applied shear force.

Consistent with the simplified envelope of interstorey drift demands on Gravity-Load Resisting Frames, given in CSA A23.3, Clause 21.11, the magnitude of the shear strain, which equals the magnitude of the resulting interstorey drift ratio, can be estimated as 60% of the inelastic rotational demand ($0.6\theta_{id}$). If the interstorey drift ratio determined by the Non-linear Dynamic Analysis is less than this value over the height of the plastic hinge, the influence of the additional interstorey drift ratios must be accounted for by separate analysis.

Sloped Columns

If the building includes inclined vertical members, the structural model must account for the vertical accelerations of all mass supported by the inclined vertical members, and must include all structural framing elements that transfer inertial forces generated by the vertical accelerations of the mass supported by the inclined vertical members.

To accurately model the coupling of the horizontal modes of the SFRS with the vertical modes of the Gravity-Load Resisting Frame, the structural model may require the complete gravity load system.

Finally, the analysis must include appropriate vertical ground motion components.

4.0 QUALITY MANAGEMENT IN PROFESSIONAL PRACTICE

4.1 ENGINEERS AND GEOSCIENTISTS BC QUALITY MANAGEMENT REQUIREMENTS

Engineering Professionals must adhere to applicable quality management requirements during all phases of the work, in accordance with the Engineers and Geoscientists BC Bylaws and quality management standards.

To meet the intent of the quality management requirements, Engineering Professionals must establish, maintain, and follow documented quality management processes for the following activities:

- Use of relevant professional practice guidelines
- Authentication of professional documents by application of the professional seal
- Direct supervision of delegated professional engineering activities
- Retention of complete project documentation
- Regular, documented checks using a written quality control process
- Documented field reviews of engineering designs and/or recommendations during implementation or construction
- Where applicable, documented independent review of structural designs prior to construction
- Where applicable, documented independent review of high-risk professional activities or work prior to implementation or construction

Engineering Professionals employed by a Registrant firm are required to follow the quality management policies and procedures implemented by the Registrant firm as per the Engineers and Geoscientists BC permit to practice program.

4.1.1 USE OF PROFESSIONAL PRACTICE GUIDELINES

Engineering Professionals are required to comply with the intent of any applicable professional practice guidelines related to the engineering work they undertake. As such, Engineering Professionals must implement and follow documented procedures to ensure they stay informed of, knowledgeable about, and meet the intent of professional practice guidelines that are relevant to their professional activities or services. These procedures should include periodic checks of the Engineers and Geoscientists BC website to ensure that the latest versions of available guidance are being used.

For more information, refer to the *Quality Management Guides – Guide to the Standard for the Use of Professional Practice Guidelines* (Engineers and Geoscientists BC 2021a), which also contains guidance for how an Engineering Professional can appropriately depart from the guidance provided in professional practice guidelines.

4.1.2 AUTHENTICATING DOCUMENTS

Engineering Professionals are required to authenticate (seal with signature and date) all documents, including electronic files, that they prepare or deliver in their professional capacity to others who will rely on the information contained in them. This applies to

documents that Engineering Professionals have personally prepared and those that others have prepared under their direct supervision. In addition, any document that is authenticated by an individual Engineering Professional must also have a permit to practice number visibly applied to the document. A permit to practice number is a unique number that a Registrant firm receives when they obtain a permit to practice engineering or geoscience in BC.

Failure to appropriately authenticate and apply the permit to practice number to Documents is a breach of the Bylaws.

For more information, refer to the *Quality Management Guides – Guide to the Standard for the Authentication of Documents* (Engineers and Geoscientists BC 2021d).

4.1.3 DIRECT SUPERVISION

Engineering Professionals are required to directly supervise any engineering work they delegate. When working under the direct supervision of an Engineering Professional, an individual may assist in performing engineering work, but they may not assume responsibility for it. Engineering Professionals who are professional licensees engineering may only directly supervise work within the scope of their licence.

When determining which aspects of the work may be appropriately delegated using the principle of direct supervision, the Engineering Professional having ultimate responsibility for that work should consider:

- the complexity of the project and the nature of the risks associated with the work;
- the training and experience of individuals to whom the work is delegated; and
- the amount of instruction, supervision, and review required.

Careful consideration must be given to delegating field reviews. Due to the complex nature of field reviews, Engineering Professionals with overall responsibility should exercise judgment when relying on delegated field observations, and should conduct a sufficient level of review to have confidence in the quality and accuracy of the field observations. When delegating field review activities, Engineering Professionals must document the field review instructions given to a subordinate. (See Section 4.1.6 Documented Field Reviews During Implementation or Construction.)

Due to the amount of work required for the design of a tall concrete building project, the Structural Engineer of Record (SER) commonly delegates work for specific components to a number of Engineering Professionals and Engineers-in-Training. The designs of the Gravity-Load Resisting Frame and the Lateral Load Resisting System are often delegated to different people, and components of each of those (e.g., columns, slabs, foundations) might even be further delegated; it is the SER's responsibility to determine which components may be delegated, and to whom, depending on each delegate's education, training, and experience. Even though the work is being delegated, the SER must still have the adequate education, training, and experience to take responsibility for all aspects of the design and be actively involved throughout the design of each component. Furthermore, the SER must oversee the design, coordination, and integration of all components, and facilitate any coordination required among delegates. Ultimately, the SER is responsible for all components of the design, regardless of whether any were delegated to others who have adequate education, training, and experience to take responsibility for the work on their own.

For more information, refer to the *Quality Management Guides – Guide to the Standard for Direct Supervision* (Engineers and Geoscientists BC 2021e).

4.1.4 RETENTION OF PROJECT DOCUMENTATION

Engineering Professionals are required to establish and maintain documented quality management processes to retain complete project documentation for a minimum of ten (10) years after the completion of a project or ten (10) years after an engineering document is no longer in use.

These obligations apply to Engineering Professionals in all sectors. Project documentation in this context includes documentation related to any ongoing

engineering work, which may not have a discrete start and end, and may occur in any sector.

Many Engineering Professionals are employed by firms, which ultimately own the project documentation. Engineering Professionals are considered compliant with this quality management requirement when reasonable steps are taken to confirm that (1) a complete set of project documentation is retained by the organizations that employ them, using means and methods consistent with the Engineers and Geoscientists BC Bylaws and quality management standards; and (2) they consistently adhere to the documented policies and procedures of their organizations while employed there.

For more information, refer to the *Quality Management Guides – Guide to the Standard for Retention of Project Documentation* (Engineers and Geoscientists BC 2021f).

4.1.5 DOCUMENTED CHECKS OF ENGINEERING WORK

Engineering Professionals are required to perform a documented quality checking process of engineering work, appropriate to the risk associated with that work. All Engineering Professionals must meet this quality management requirement.

The checking process should be comprehensive and address all stages of the execution of the engineering work. This process would normally involve an internal check by another Engineering Professional within the same organization. Where an appropriate internal checker is not available, an external checker (i.e., one outside the organization) must be engaged. In some instances, self-checking may be appropriate. Where internal, external, or self-checking has been carried out, the details of the check must be documented. The documented quality checking process must include checks of all professional deliverables before being finalized and delivered.

Engineering Professionals are responsible for ensuring that the checks being performed are appropriate to the level of risk associated with the item being checked.

Considerations for the level of checking should include:

- the type of item being checked;
- the complexity of the subject matter and underlying conditions related to the item;
- the quality and reliability of associated background information, field data, and elements at risk; and
- the Engineering Professional's training and experience.

As determined by the Engineering Professional, the individual doing the checking must have current expertise in the discipline of the type of work being checked, be sufficiently experienced and have the required knowledge to identify the elements to be checked, be objective and diligent in recording observations, and understand the checking process and input requirements.

Considering software is used extensively in the design of tall concrete buildings and, in many cases, a high number of delegates are involved in these projects, it is critical that the SER be actively involved in making and/or reviewing the assumptions, input, and output of all models used. The use of technology (i.e., spreadsheets, analysis and modelling software) is integral to the design of tall concrete buildings. The SER must use professional judgment to continually question whether the results make sense and, as required, must perform independent analyses with different software, simplified models, or hand calculations to verify the results.

Modelling considerations for the Gravity-Load Resisting Frame, the Lateral Force Resisting System for wind loads, the Lateral Force Resisting System for seismic loads (i.e., the Seismic Force Resisting System), and Non-linear Dynamic Analysis are discussed in Section 3.2 Design for Gravity Loads, Section 3.3.4 Modelling Considerations, Section 3.4.3.1 Modelling Requirements, and Section 3.4.8.2 Modelling Considerations, respectively.

For more information, refer to the *Quality Management Guides – Guide to the Standard for Documented Checks of Engineering and Geoscience Work* (Engineers and Geoscientists BC 2021g).

4.1.6 DOCUMENTED FIELD REVIEWS DURING IMPLEMENTATION OR CONSTRUCTION

Field reviews are reviews conducted at the site of the construction or implementation of the engineering work. They are carried out by an Engineering Professional or a subordinate acting under the Engineering Professional's direct supervision (see Section 4.1.3 Direct Supervision).

Field reviews enable the Engineering Professional to ascertain whether the construction or implementation of the work substantially complies in all material respects with the engineering concepts or intent reflected in the engineering documents prepared for the work.

It is important to verify during construction that all assumptions and limitations made during design are being appropriately implemented on site. The performance of the building can be significantly and negatively affected by improper implementation of the design requirements.

During field reviews, a number of items related to concrete walls, including shear walls, core walls, and columns, require careful review. These items may include, but are not limited to:

- cover to wall reinforcement, taking into account the applicable environmental and fire exposure;
- cover, spacing, and geometry of ties;
- grade, size, spacing, and lap splices of reinforcement;
- anchorage of horizontal reinforcement (to tie the zones together);
- location of penetrations, ducts, and conduits, particularly those in and around the Lateral Force Resisting System elements; and
- pour height of column and wall concrete is to the underside of the slab above, to not detract from the punching shear capacity of the slab.

Slabs with sloping for drainage, such as exterior or parking slabs, are another example of critical components to review on site for conformance with structural drawings. For example, if the thickness and placement of reinforcement in the slab is not as expected, the slab will not perform as expected. Other examples of critical items to be verified during field reviews of slabs include, but are not limited to, the following:

- The thickness of concrete slabs is as designed, since increased thickness may overload the supporting columns and decreased thickness may cause punching shear, strength, or deflection issues.
- The high and low spots of the concrete slab are as designed.
- Whether the concrete slab itself is sloped (bottom and/or top) or whether topping will subsequently be added.
- Whether the reinforcement is placed in the minimum slab depth or is sloped with the topping (only applies when the concrete is placed monolithically).
- The cover on the reinforcement and the arrangement of reinforcement layers are as designed.
- Whether any "puddling" of higher-strength concrete or use of high-strength reinforcement are implemented as designed.
- The locations of penetrations, ducts, and conduits, particularly those in transfer slabs or beams or those in close proximity to columns, are as designed.
- The location and detailing of pour joints are as designed.
- Ties are in the correct place in the raft (i.e., not on top of the bottom reinforcement).

The above lists of critical items for field review do not remove the requirement for the SER, or a person under the SER's direct supervision, to review all components and details of the structural design.

For more information, refer to the *Quality Management Guides – Guide to the Standard for Documented Field Reviews During Implementation or Construction* (Engineers and Geoscientists BC 2021h).

4.1.7 DOCUMENTED INDEPENDENT REVIEW OF STRUCTURAL DESIGNS

Engineering Professionals developing structural designs are required to engage an independent review of their structural designs. An independent review is a documented evaluation of the structural design concept, details, and documentation based on a qualitative examination of the substantially complete structural design documents, which occurs before those documents are issued for construction or implementation. It is carried out by an experienced Engineering Professional qualified to practice structural engineering, who has not been involved in preparing the design.

The Registered Professional of Record (i.e., the SER) must conduct a risk assessment after conceptual design and before detailed design to (1) determine the appropriate frequency of the independent review(s); and (2) determine if it is appropriate for the independent reviewer to be employed by the same firm as the Registered Professional of Record (Type 1 independent review), or if the independent reviewer should be employed by a different firm (Type 2 independent review).

The risk assessment may determine that staged reviews are appropriate; however, the final independent review must be completed after checking has been completed and before the documents are issued for construction or implementation. Construction must not proceed on any portion of the structure until an independent review of that portion has been completed.

The documented risk assessment used to determine whether a Type 1 or Type 2 independent review is required should consider:

- the consequences of failure;
- the complexity of the structural design;
- the modes of failure;
- the nature of the design assumptions;
- the uniqueness of the structural design; and
- whether a substantially similar structural design by the same Registered Professional of Record was subjected to a Type 2 independent review in the reasonably proximate past.

Tall concrete buildings are generally large buildings with many occupants, and thus the consequences of a failure are potentially dire. Therefore, it is essential to accurately assess whether a Type 1 or Type 2 independent review is required.

A Type 1 independent review may be appropriate for a tall concrete building project when the SER and the firm have previous experience with similar buildings, and one of those buildings that was substantially similar was recently subjected to a Type 2 independent review.

Type 2 independent reviews are appropriate when the SER and the firm do not have experience with similar buildings. Many tall concrete buildings have unique architecture, which increases the possibility that the structural system itself, and the complexity of the structural design, will also be unique. Type 2 independent reviews are recommended for tall concrete buildings incorporating any irregular or uncommon characteristics, or those with a structural design involving issues without well-defined solutions. For example, the design of a building with multiple towers on a shared large podium structure would present a problem without a well-defined solution available in the Code.

In addition, when Non-linear Dynamic Analysis is used for the seismic design, both a Type 2 independent review and a Peer Review are required. Requirements for Peer Review are separate from those for documented independent review of structural designs. See Section 4.3 Peer Review for more information.

One reason Type 2 independent reviews are generally recommended for tall concrete buildings is that information about what is considered good professional practice is evolving more rapidly than the adoption of new editions of the Code, particularly with regard to design for earthquake ground motions. A related issue is the prevalence of unusual irregularities in tall

concrete buildings; i.e., significant irregularities that are not accounted for in the current edition of the Code.

While Engineers and Geoscientists BC requires that the independent review be completed before the structural documents are issued for construction or implementation, some Authorities Having Jurisdiction (AHJs) require that the independent review be conducted before the structural documents are issued for building permit. Regardless of whether an independent review is required before a building permit, the structural documents must be substantially complete such that the design can be checked for conformance to the Code. See the *Practice Advisory – Issued for Building Permit Documents* (Engineers and Geoscientists BC 2020c) for more information.

If the AHJ requires that the independent review be completed before a building permit, regardless of whether significant changes subsequently occur, the SER should consider requesting a follow-up independent review from the independent reviewer closer to when the structural documents are to be issued for construction.

For more information, refer to the *Quality Management Guides – Guide to the Standard for Documented Independent Review of Structural Designs* (Engineers and Geoscientists BC 2021i).

4.1.8 DOCUMENTED INDEPENDENT REVIEW OF HIGH-RISK PROFESSIONAL ACTIVITIES OR WORK

Engineering Professionals must perform a documented risk assessment prior to initiation of a professional activity or work, to determine if that activity or work is high risk and requires a documented independent review.

If the activities or work are deemed high risk, and an independent review is required, the results of the risk assessment must be used to (1) determine the appropriate frequency of the independent review(s); and (2) determine if it is appropriate for the independent reviewer to be employed by the same firm as the Registered Professional of Record, or if the independent reviewer should be employed by a different firm.

The documented independent review of high-risk professional activities or work must be carried out by an Engineering Professional with appropriate experience in the type and scale of the activity or work being reviewed, who has not been involved in preparing the design.

The documented independent review must occur prior to implementation or construction; that is, before the professional activity or work is submitted to those who will be relying on it.

For more information, refer to the *Quality Management Guides – Guide to the Standard for Documented Independent Review of High-Risk Activities or Work* (Engineers and Geoscientists BC 2021j).

4.2 OTHER QUALITY MANAGEMENT REQUIREMENTS

Engineering Professionals must also be aware of any additional quality management requirements from other sources that are relevant to their work, which may include but are not limited to:

- legislation and regulations at the local, regional, provincial, and federal levels;
- policies of AHJs at the local, regional, provincial, and federal levels;
- agreements and service contracts between clients and Engineering Professionals or their firms; and/or
- standards for engineering firms, particularly those that apply to quality management system certification, such as the ISO 9000 family.

Engineering Professionals should assess any areas of overlap between the Engineers and Geoscientists BC quality management requirements and the requirements of other applicable sources. If the requirements of different sources overlap, Engineering Professionals should attempt to meet the complete intent of all requirements.

Where there are conflicts between requirements, Engineering Professionals should negotiate changes or waivers to any contractual or organizational requirements which may conflict with requirements of legislation, regulation or the Engineers and Geoscientists BC Code of Ethics. Generally, no contractual obligation or organizational policy that may apply to an Engineering Professional will provide justification or excuse for breach of any of the Engineering Professional's obligations under any legislation, regulation, or the Engineers and Geoscientists BC Code of Ethics. Where such conflicts arise and cannot be resolved, Engineering Professionals should consider seeking legal advice from their own legal advisers on their legal rights and obligations in the circumstances of the conflict, and they may also seek practice advice from Engineering and Geoscientists BC on any related ethical dilemma that they may face in the circumstances. See Section 4.4 Practice Advice for contact information.

4.3 PEER REVIEW

A Peer Review of tall concrete building design may be required for a number of reasons. Peer Review may be a requirement of the Code and its referenced documents, or of legislation or regulations; or it may be required by the client, the AHJ, or another party.

As described in Section 3.4.8 Evaluation of Life Safety Performance Using Non-linear Dynamic Analysis of these guidelines, when Non-linear Dynamic Analysis is used for the seismic design of a concrete building designed to CSA A23.3, Design of Concrete Structures, "the non-linear analysis and resulting design shall be reviewed by a qualified independent review panel." This mandatory requirement is clearly stated in CSA A23.3, Clause 21.2.3.

In turn, CSA A23.3 refers to the Los Angeles Tall Buildings Structural Design Council guidelines for the composition of a qualified independent Peer Review panel. The Peer Review panel must include at least three reviewers, including at least one reviewer with recognized expertise in each of the following areas: earthquake-resistant design, Non-linear Dynamic Analysis, and seismic hazard assessment. For tall concrete building projects in BC, it is recommended that at least one reviewer on the Peer Review panel be a Registrant of Engineers and Geoscientists BC.

Peer Review is a requirement under the Code, while independent review of structural design is a mandatory requirement under the Engineers and Geoscientists BC Bylaws; therefore, when applicable, a Peer Review must be done in addition to a documented independent review of structural designs. An independent review is a documented evaluation of the structural design concept, details, and documentation based on a qualitative examination of the substantially complete structural design documents, which occurs before those documents are issued for construction or implementation. It is carried out by an experienced Engineering Professional qualified to practice structural engineering, who has not been involved in preparing the design. See Section 4.1.7 Documented Independent Review of Structural Designs for more information.

The reasons for using Non-linear Dynamic Analysis for the seismic design of a concrete building (with a Peer Review) will generally also trigger the requirement for a Type 2 independent review of structural design. As described in Section 4.1.7 of these guidelines, building design with any irregular or uncommon aspects (such as those requiring Non-linear Dynamic Analysis), or where the structural design involves issues without well-defined solutions (such as is the case with Non-linear Dynamic Analysis), are reasons for a Type 2 independent review.

The responsibilities for independent review of structural design cannot be divided up and assigned to different professionals; one Engineering Professional must take responsibility for the complete independent review of the structural design. For example, the Peer Review panel of independent subject matter experts from outside the SER's firm would review aspects of the building design requiring Non-linear Dynamic Analysis. But although the remaining aspects of the structural

design may seem to meet the requirements for only a Type 1 independent review, it would not be appropriate to divide the responsibility for an independent review of structural design in this way (i.e., by separating the independent reviews of the Lateral Force Resisting System and the Gravity-Load Resisting Frame).

The SER is responsible for ensuring that both a documented independent review of structural designs, as per the Bylaws, and a full Peer Review, as per the Code for projects requiring Non-linear Dynamic Analysis, are completed during the design of a tall concrete building, before the documents are issued for construction.

The Peer Review panel must review and approve the Basis of Design Document before the SER commences the detailed design of the building; as such, the Peer Review panel should be engaged as early as possible in the design process.

Engineering Professionals involved in the Peer Review of a tall concrete building must adhere to the Engineers and Geoscientists BC Code of Ethics, specifically Principle 13, which requires that Registrants conduct themselves with fairness, courtesy, and good faith towards clients, colleagues, and others, give credit where it is due, and accept, as well as give, honest and fair professional comment. Engineering Professionals should inform (or make every effort to inform) the Registered Professionals of Record when applicable, prior to reviewing their work. See also the *Guide to the Code of Ethics*, Section 4.13.5 Work Reviews (Engineers and Geoscientists BC 2021k).

4.4 PRACTICE ADVICE

Engineers and Geoscientists BC provides their Registrants and others with assistance addressing inquiries related to professional practice and ethics.

Practice advisors at Engineers and Geoscientists BC can answer questions regarding the intent or application of the professional practice or quality management aspects of these guidelines.

To contact a practice advisor, email Engineers and Geoscientists BC at practiceadvisor@egbc.ca.

5.0 PROFESSIONAL REGISTRATION & EDUCATION, TRAINING, AND EXPERIENCE

5.1 PROFESSIONAL REGISTRATION

Engineering Professionals have met minimum education, experience, and character requirements for admission to their professions. However, the educational and experience requirements for professional registration do not necessarily constitute an appropriate combination of education and experience for structural engineering services for tall concrete building projects. Professional registration alone does not automatically qualify an Engineering Professional to take professional responsibility for all types and levels of professional services in this professional activity.

It is the responsibility of Engineering Professionals to determine whether they are qualified by training and/or experience to undertake and accept responsibility for carrying out structural engineering services for tall concrete building projects (Code of Ethics Principle 2).

5.2 EDUCATION, TRAINING, AND EXPERIENCE

Structural engineering services for tall concrete building projects, as described in these guidelines, requires minimum levels of education, training, and experience in many overlapping areas of engineering.

Engineering Professionals who take responsibility for the integrity of the structural systems of tall concrete building projects must adhere to the second principle of the Engineers and Geoscientists BC Code of Ethics, which is to “practice only in those fields where training and ability make the registrant professionally competent” and, therefore, must evaluate their own qualifications and must possess the appropriate education, training, and experience to provide the services.

The level of education, training, and experience required of Engineering Professionals should be adequate for the complexity of the project. This section describes indicators that Engineering Professionals can use to determine whether they have an appropriate combination of education and experience.

Note that these indicators are not an exhaustive list of education and experience types that are relevant to structural engineering services for tall concrete building projects. Satisfying one or more of these indicators does not automatically indicate adequate competence in this area of practice.

Beyond the Code of Ethics and Bylaw requirements of Engineers and Geoscientists BC, some Authorities Having Jurisdiction (AHJ) stipulate that only a Struct.Eng. can take professional responsibility for structural engineering services on certain types of buildings. Engineering Professionals should review any registration requirements of the AHJ before taking on the structural engineering services for a tall concrete building project.

5.2.1 EDUCATIONAL INDICATORS

Certain indicators show that Engineering Professionals have received education that might qualify them to participate professionally in the structural design and related services for tall concrete building projects. Educational indicators are subdivided into formal education (such as university or engineering school) and informal education (such as continuing education).

Formal educational indicators include having obtained or completed one or more of the following:

- An undergraduate-level degree in civil engineering or a related engineering field from an accredited engineering program
- A postgraduate-level degree in structural engineering or a related engineering field from an accredited engineering program

Informal educational indicators include having participated in or undertaken one or more of the following related to structural engineering design or specifically related to tall concrete building projects:

- Training courses facilitated by the Engineering Professional's employer
- Continuing education courses or sessions offered by professional organizations (such as Engineers and Geoscientists BC or the Structural Engineers Association of British Columbia)
- Conferences or industry events
- A rigorous and documented self-study program involving a structured approach that contains materials from textbooks and technical papers

5.2.2 EXPERIENCE INDICATORS

Certain indicators show that Engineering Professionals have an appropriate combination of experience that might qualify them to participate professionally in structural engineering design and related services for tall concrete building projects. To take full responsibility for a tall concrete building project as the Structural Engineer of Record (SER), experience indicators include having completed one or more of the following:

- Has significant experience as a senior Engineering Professional, under the direct supervision of the SER, participating in the analysis and design of Lateral Force Resisting Systems and Gravity-Load Resisting Frames of tall concrete buildings, including the coordination and integration of the two systems
- Was the SER responsible for the analysis and design of Lateral Force Resisting Systems and Gravity-Load Resisting Frames of low- or mid-rise concrete buildings
- Participated in academic or industry working groups that focus on the analysis and design of tall concrete buildings

6.0 REFERENCES

Documents cited in these guidelines appear in Section 6.1 Legislation; Section 6.2 References; and Section 6.3 Codes and Standards.

6.1 LEGISLATION

The following legislation is referenced in these guidelines:

Professional Governance Act [SBC 2018], Chapter 47.

6.2 REFERENCES

The following documents are referenced in these guidelines:

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6.3 CODES AND STANDARDS

The following codes and standards are referenced in these guidelines:

ASCE 41, Seismic Evaluation and Retrofit of Existing Buildings.

ACI 318, Building Code Requirements for Structural Concrete.

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CSA A23.3:19, Design of Concrete Structures.

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7.0 APPENDICES

APPENDIX A: AUTHORS AND REVIEWERS

All contributors are presented in alphabetical order by last name within their respective sections.

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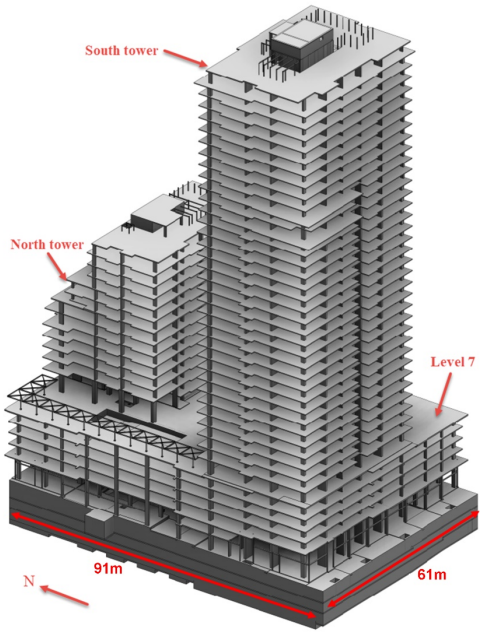
APPENDIX B: NON-LINEAR DYNAMIC ANALYSIS EXAMPLES

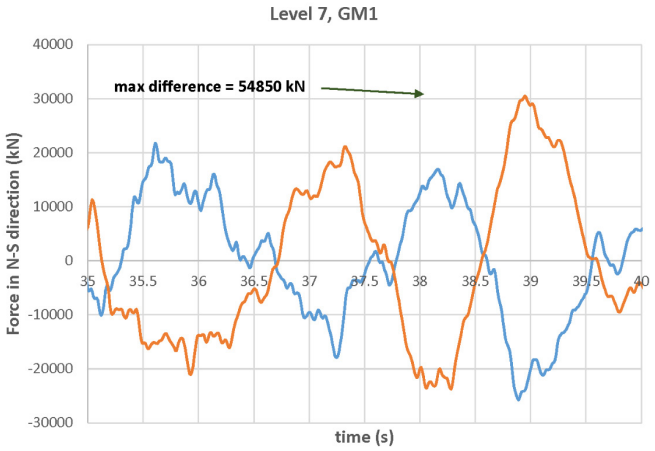
APPENDIX B: NON-LINEAR DYNAMIC ANALYSIS EXAMPLES

EXAMPLE 1: TWO TOWERS ON A COMMON PODIUM

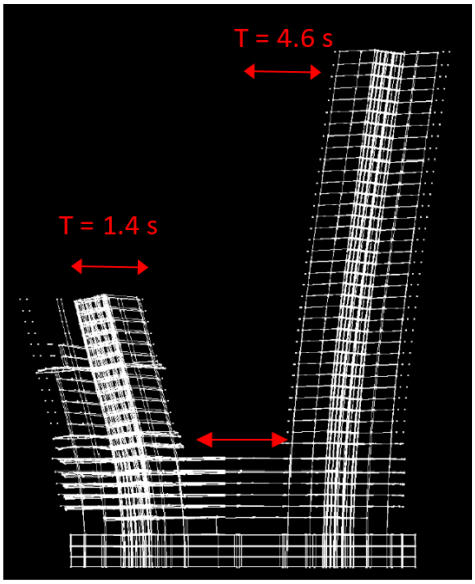
- Non-linear Dynamic Analysis was used to determine the transfer forces that must be resisted by the diaphragms connecting two towers that share a common podium structure.
- Figure A and Figure C show how a 42-storey tower is connected to a 21-storey tower from the foundation to level 7. The two towers have very different modal properties, with the first three modal periods of the taller tower exceeding the first translational period of the shorter tower.
- Figure B shows the results at level 7 for one ground motion, with the maximum value occurring at T = 38.9 s. Average transfer force for 11 pairs of ground motions is 65,000 kN, which is more than twice as much as the design base shear. The diaphragm at this level must be designed to resist
  1) tension/compression force; and
  2) the shear force from out of phase movement of the two towers.



(A) Three-dimensional view of building



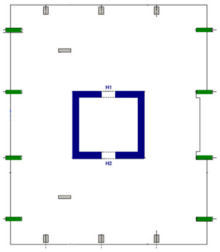
(B)



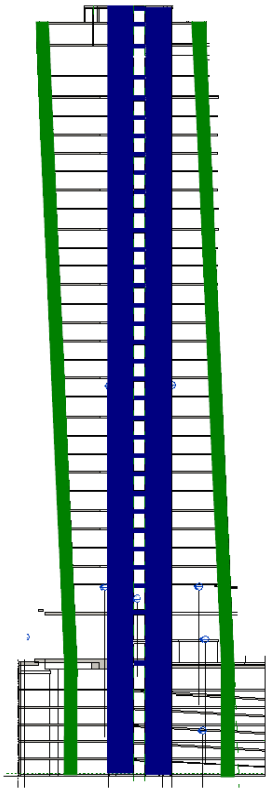
(C)

EXAMPLE 2: GRAVITY-INDUCED LATERAL DEMAND (GILD)

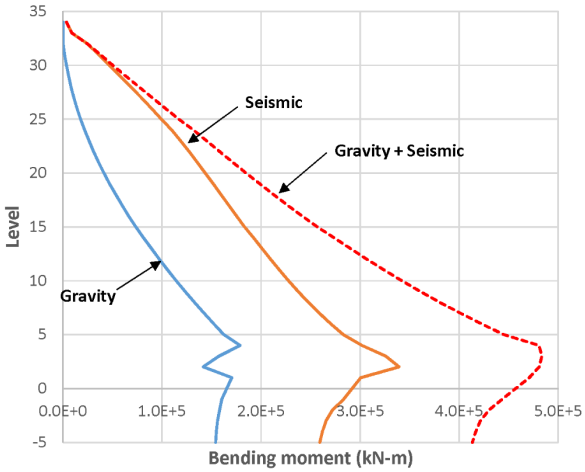
- The building (Figure A and Figure B) has eight sloping columns (highlighted in green).
- In the direction of the GILD, the SFRS is two C-shape wall piers connected by coupling beams to form a ductile coupled wall system (Figure A and Figure B).
- The ratio of the bending moment demand from the GILD to the bending moment demand from seismic load combination is 0.56 at the grade level (Figure C).
- When a building has such a large gravity-induced lateral demand, the Code requires Non-linear Dynamic Analysis be used to determine that the design of the building is acceptable.
- The tendency of the building to lean in the direction of the gravity forces is observed in the coupling beam rotational demands and the storey drifts (Figure D).
- The mean of maximum inelastic rotational demand and storey drift is considerably larger in the direction of gravity loads (Figure E).



(A)



(B) Elevation of the tower



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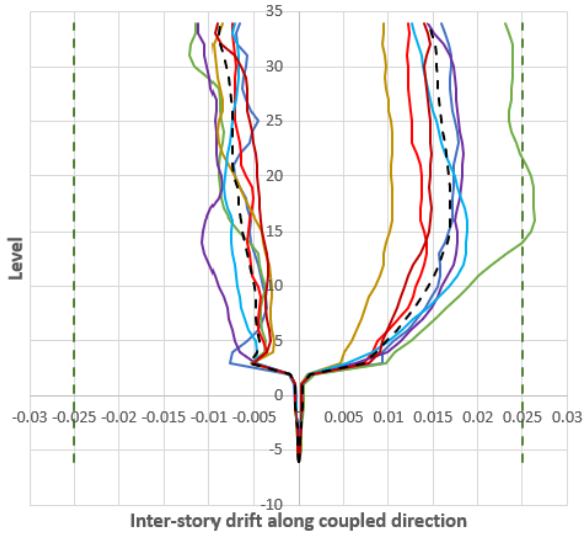
EXAMPLE 2: GRAVITY-INDUCED LATERAL DEMAND (GILD) (continued)



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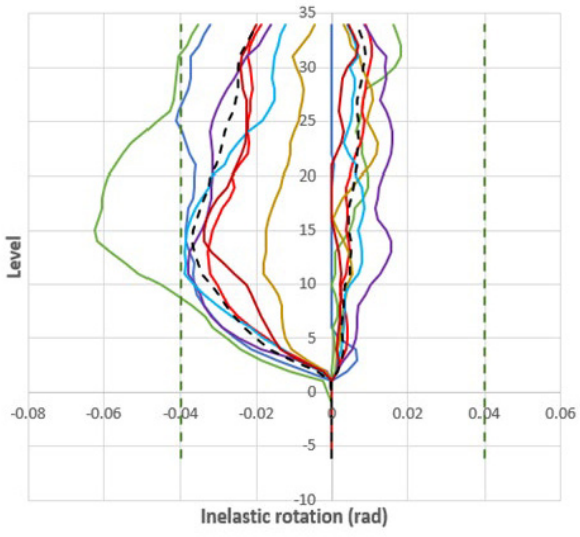
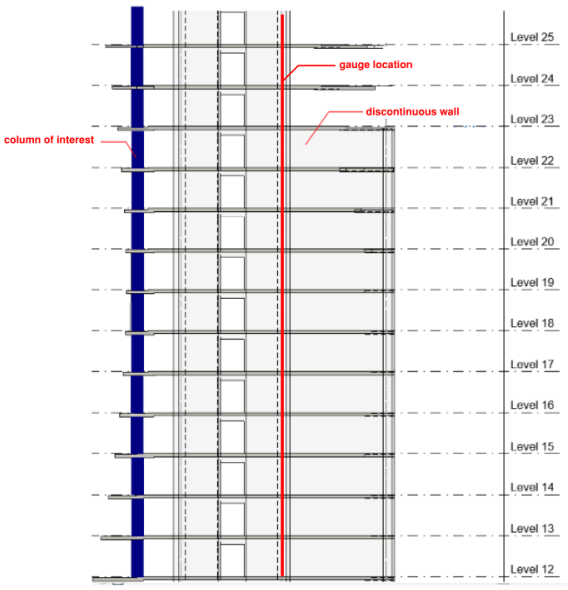


(E)

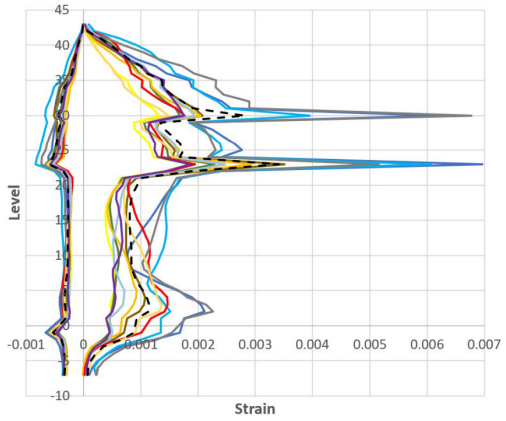
APPENDIX B: NON-LINEAR DYNAMIC ANALYSIS EXAMPLES

EXAMPLE 3: DISCONTINUOUS SHEAR WALL

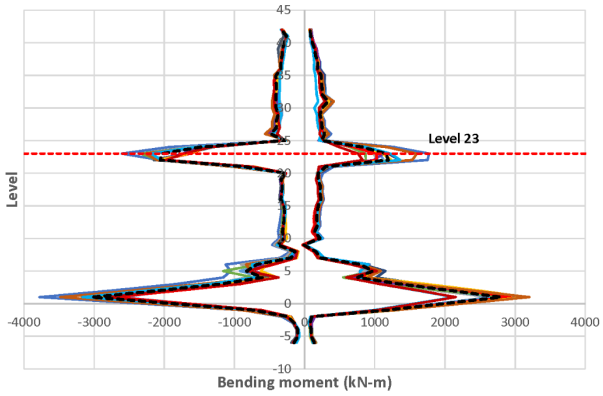
- Non-linear Dynamic Analysis demonstrates the reason for the special detailing required in shear walls and gravity-load columns at every location of a Type 1 irregularity.
- The partial elevation (Figure A) shows a shear wall terminating at Level 23. The vertical strains at the end of the wall were determined using “strain gauges” in the non-linear analysis model, the location of which is indicated by the vertical red line in Figure A.
- As shown in Figure B, large compression strains occur in the wall immediately above the discontinuity at Level 23, and at Level 30 due to a change in the size of the boundary element at the end of the wall.
- The larger curvature demands in the core immediately above Level 23 induce larger bending moments in the gravity columns as shown in Figure C.



(A)



(B)

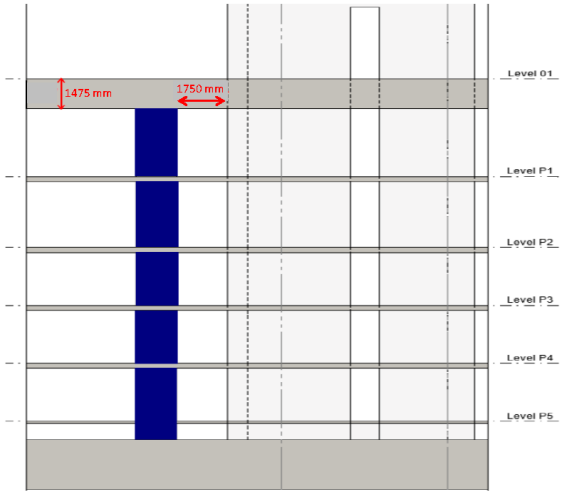


(C)

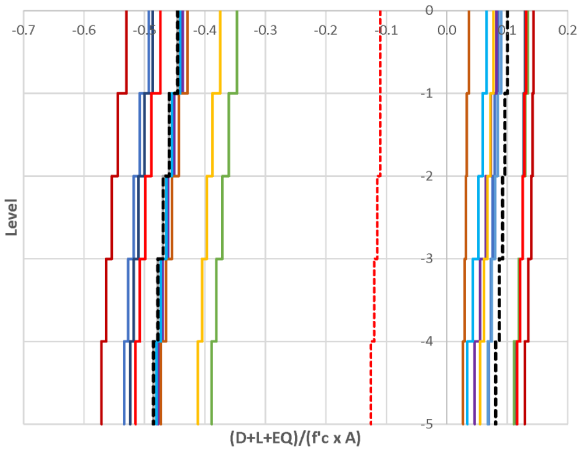
APPENDIX B: NON-LINEAR DYNAMIC ANALYSIS EXAMPLES

EXAMPLE 4: COLUMN IN CLOSE PROXIMITY TO CORE

- A gravity-load column, located less than 2 m from the core, supports a 1475 mm thick transfer slab that is connected to the core at grade level (Figure A).
- Non-linear Dynamic Analysis was used to investigate the demands on the column due to the transfer slab acting as an outrigger.
- The axial force induced in the column is more than four times the axial force from dead plus live load; thus the column will be subjected to axial tension in one direction of the earthquake and high compression in the other direction (Figure B).
- In addition, bending moments (Figure C) and shear forces (Figure D) are induced into the gravity-load column.



(A) Elevation of core wall and column at parking levels



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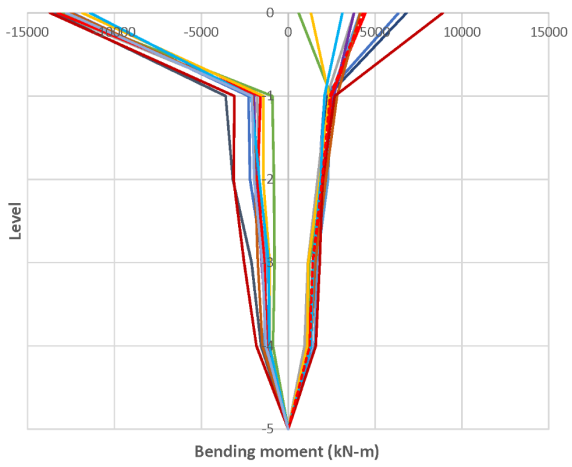

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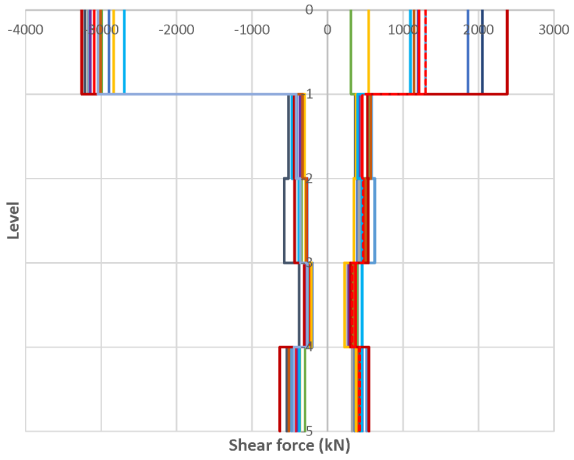
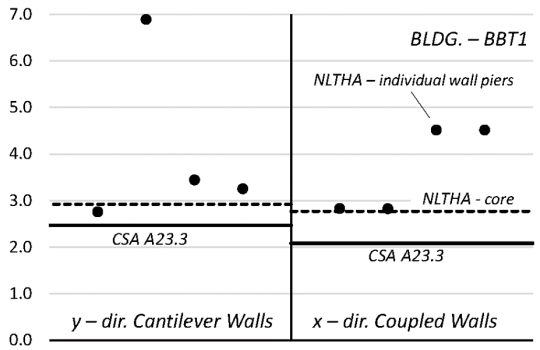


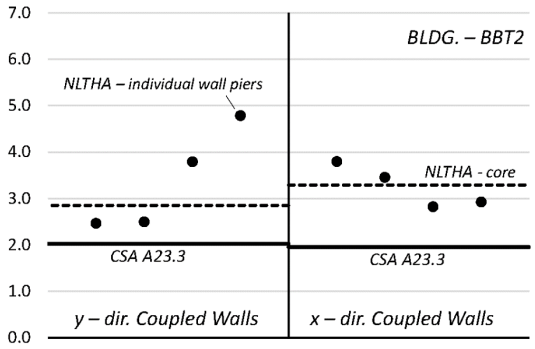
(D)

EXAMPLE 5: INELASTIC EFFECTS OF HIGHER MODE SHEARS

- The plots below (Figures A and B) compare the mean shear force demands determined using Non-linear Dynamic Analysis and the design shear force demands specified by CSA A23.3, as a ratio of the shear demands determined from Linear Dynamic Analysis of the building using the force reduction factors specified by the Code.
- The mean shear demands on the total core from Non-linear Dynamic Analysis is shown as dashed lines; the mean shear demands on individual wall piers is shown as dots; and shear force demands specified by CSA A23.3 are shown as solid lines.
- Separate plots are provided in the two directions of the core. The data in the Figure A is from a building that is similar to a typical core wall building in BC, with coupled walls in one direction and cantilever walls in the perpendicular direction, while the data in the Figure B is from a building that has coupled walls in two directions.
- The shear force demands from Non-linear Dynamic Analysis are the single largest shear force pulse existing in the structure (for a short time) during the ground motion.
- The design shear force demands specified by CSA A23.3 include a flexural overstrength factor and a dynamic amplification factor for higher modes in the cantilever wall direction.
- As discussed in Section 3.4.4.5 Design of Walls for Shear of these guidelines, the dynamic amplification in CSA A23.3 is a lower-bound value because the maximum shear force occurs only once during an earthquake and lasts for a very short time; well-detailed concrete walls have shear ductility; and the maximum shear force generally does not occur when the base rotation is maximum, while the CSA A23.3 shear design procedures for concrete walls assumes that it does.
- Non-linear Dynamic Analysis indicates different amplification factors for different walls; however, it is important to note that the redistribution of shear forces to individual wall piers in a core depend very much on the local wall shear strains, which are typically not modelled accurately in Non-linear Dynamic Analysis.



(A)



(B)

APPENDIX C: SEISMICITY IN SOUTHWESTERN BRITISH COLUMBIA

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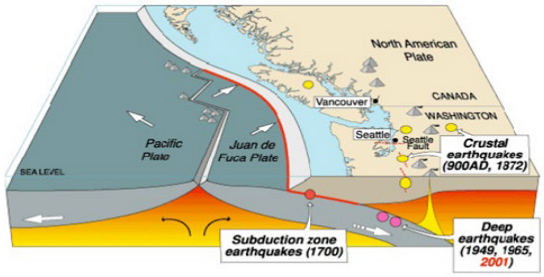
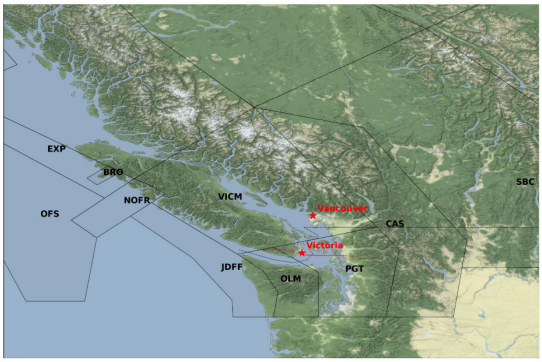
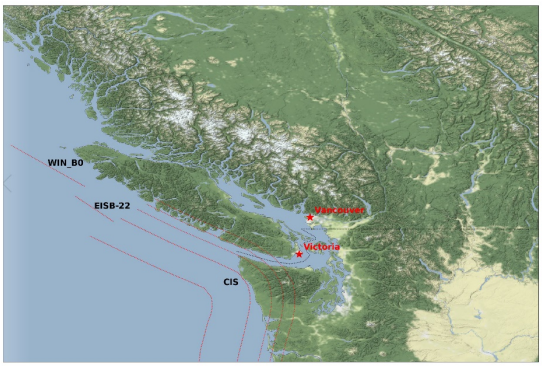


- Southwestern British Columbia (BC) has a unique seismic setting that includes earthquakes from three sources:
  - crustal events, which occur along shallow faults in the earth's crust;
  - in-slab events, which occur deep within subducting tectonic plates; and
  - subduction interface events, which are caused by slip between subducting tectonic plates.
- (Figure from the United States Geological Survey; public domain)



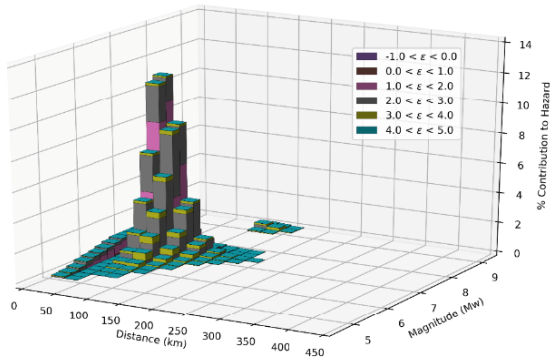
- Several areal crustal sources impact the seismic hazard in Southwestern BC.
- The Geological Survey of Canada has included active faults through to the western coast of Vancouver Island for the *National Building Code of Canada* (*NBC*) 2020, including Devil's Mountain Fault and a west-northwestwards extension of this fault called the Leech River Fault, incorporating various fault lengths and rates (Adams et al. 2019, Halchuk et al. 2019, and Open File 8630).

- Two in-slab area sources are included in the model to capture the seismicity that occurs in the subducting Juan de Fuca plate including:
  - the Onshore Juan de Fuca Plate Bending; and
  - the Georgia Straight/Puget Sound.
- The Georgia Straight/Puget Sound was refined in the Canada's 6th generation seismic hazard model to be used in the *NBC 2020*. The 2015 Georgia Straight/Puget Sound was replaced by three sources (western, central, and eastern) set at 50 km, 55 km, and 60 km depths to better model the dip of the in-slab source (Adams et al. 2019, Open File 8630).



- The Juan de Fuca subduction zone was modelled with a series of finite fault models, which include the Juan de Fuca segment of the Cascadia subduction zone (termed Cascadia Interface Subduction or CIS source) and the Explorer segment of the Cascadia subduction zone (termed Explorer Interface Subduction source).
- Also included along with the interface sources are two thrust fault sources: the Winona Thrust Fault and Haida Gwaii Thrust Fault.

APPENDIX C: SEISMICITY IN SOUTHWESTERN BRITISH COLUMBIA



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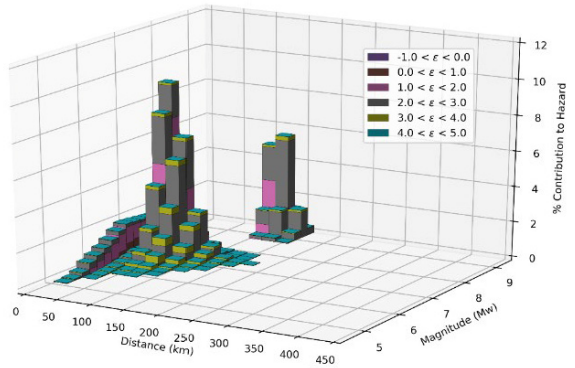
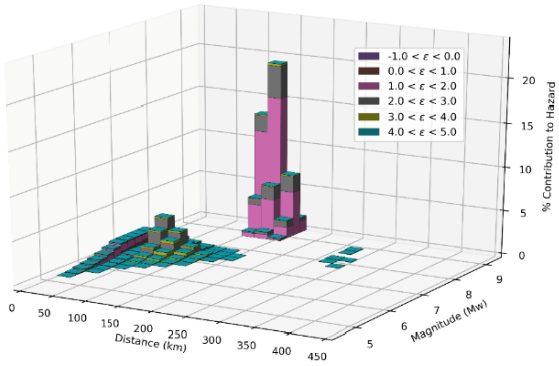
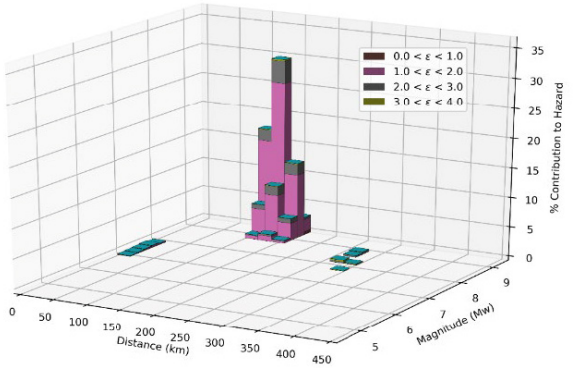


S_a = 0.772g at T= 0.5 sec



S_a = 0.442g at T= 1.0 sec



S_a = 0.075g at T= 5.0 sec

- The disaggregation results in Vancouver, British Columbia, Canada, V_{s30}= 450 m/s using sixth generation of hazard models show that shallow crustal earthquakes with a magnitude of 5 to 7.5 at a distance of less than 30 km have minor contribution at short periods (peak ground acceleration [PGA], up to 0.8 sec).

- A large in-slab contribution is observed from magnitude 6 to 7.5 earthquakes at distances of 50 to 150 km from PGA up to T= 1~2 seconds.
- A significant interface (subduction) contribution is seen from magnitude 8 to 9 earthquakes at distances of 100 to 150 km for the period of 1.0 second and greater.

NOTES:

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