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BC Guidelines for Seismic Design of Tall Concrete Buildings Using Nonlinear Dynamic Analysis

Perry Adebar,[1*] Ehsan Dezhdar[2] and Armin Bebamzadeh[3]

[1]Professor, Department of Civil Engineering, The University of British Columbia, Vancouver, BC, Canada
[2]Senior Associate, Glotman-Simpson Consulting Engineers, Vancouver, BC, Canada
[3]Research Associate and Instructor, Dept. of Civil Engineering, The University of British Columbia, Vancouver, BC, Canada
[*]adebar@civil.ubc.ca (Corresponding Author)

ABSTRACT

Highrise buildings in British Columbia (BC), which are invariably concrete shear (core) wall buildings, are normally designed using linear (modal response spectrum) analysis and the prescriptive requirements in the National Building Code of Canada (NBC) and Canadian Standard CSA A23.3. In comparison to recent practice on the west coast of the United States, nonlinear dynamic (time history) analysis has rarely been used for the design of highrise core wall buildings in BC. As building architecture has recently become more unique and less like the buildings that the prescriptive procedures were developed for, the need to use nonlinear time history analysis (NLTHA) to confirm the life safety performance of buildings has increased. While excellent guidelines for the seismic design of buildings using NLTHA are available from the United States, these guidelines do not result in designs that are consistent with the Canadian prescriptive design requirements. Thus, specific guidelines were developed for use in BC. The guidelines were recently published as part of an Engineers and Geoscientists of BC (EGBC) professional practice guideline *Structural Engineering Services for Tall Concrete Building Projects*. This paper provides a high-level summary of the guidelines, which were developed by the authors, explains the rationale behind the development, and provides a discussion on some of the differences between the BC guidelines and the commonly used Los Angeles (LATBSDC) guidelines. Specifically, a detailed comparison on the shear design requirements for concrete walls shows that while the procedures are very different, the resulting designs are very similar. Information is also presented about expected changes and additions to the next edition of the BC guidelines, including the addition of serviceability evaluation of highrise concrete buildings consistent with the latest editions of NBC, the addition of design guidelines for outriggers, and the use of NLTHA of wind demands to permit the redistribution of wind force demands on coupling beams.

Keywords: Concrete buildings; Improvements to seismic design; Nonlinear analysis; Structural engineering practice

INTRODUCTION

The seismic design of tall concrete buildings in British Columbia (BC) must meet the requirements of the National Building Code of Canada (NBC), which is adopted provincially as the British Columbia Building Code (BCBC), and the Canadian Standard CSA A23.3-14, Design of Concrete Structures (referred to here as simply CSA A23.3). In British Columbia (and most of Canada), tall concrete buildings are shear wall buildings, and in high seismic regions such as Vancouver and Victoria BC, tall concrete buildings are invariably core wall buildings with the shear walls arranged in a central core.

Clause 21 of CSA A23.3 has displacement-based procedures for the seismic design of concrete shear wall buildings. Modal response spectrum analysis and an appropriate effective flexural stiffness accounting for the level of damage is used to make accurate estimates of the top wall displacement. Simplified mechanics-based procedures are then used to estimate inelastic deformation demands, such as plastic hinge rotations, from top wall displacement. Until recently, nonlinear dynamic (time history) analysis has rarely been used to design highrise concrete buildings in BC. As building architecture has become more exciting and less structurally regular in recent years, the need to use nonlinear time history analysis (NLTHA) to confirm the life safety performance of structurally irregular buildings in British Columbia has increased.

NLTHA is an effective tool for extrapolating the simplified (prescriptive) seismic design procedures given in CSA A23.3 to different structural forms than was assumed when developing the prescriptive procedures. An example is two building towers sharing a common podium structure. NLTHA can be used to make accurate estimates of the transfer forces that must be resisted by the diaphragms connecting the two towers. Generally, when NLTHA is applied to the regular "cookie cutter" buildings that the procedures in CSA A23.3 were developed, the designs using NLTHA should give similar results as the prescriptive procedures. NLTHA can also reveal information that is not adequately addressed in the prescriptive procedures. For example, NLTHA may give much higher floor accelerations than are determined using linear dynamic analysis procedures.

There are two excellent guidelines currently available for the seismic design of tall concrete buildings using NLTHA – the Pacific Earthquake Engineering Center Tall Buildings Initiative (PEER TBI) guidelines [1] and the Los Angeles Tall Buildings Structural Design Council (LATBSDC) guidelines [2]. These guidelines provide requirements consistent with two US Codes *Minimum Design Loads for Buildings and Other Structures* (ASCE/SEI 7), and *Building Code Requirements for Structural Concrete* (ACI 318). Thus, certain aspects of these guidelines are not appropriate for use in the design of tall concrete buildings in British Columbia as they will not result in the same design as the required by NBC and CSA A23.3.

Engineers and Geoscientists of BC (EGBC) recently published a professional practice guideline entitled *Structural Engineering Services for Tall Concrete Building Projects*. [3] That document includes Section 3.4.8 entitled *Evaluation of Life Safety Performance Using Non-Linear Dynamic Analysis* written by the authors of this paper. Section 3.4.8 of the EGBC professional practice guideline is herein referred to as the BC Guidelines for seismic design of tall concrete buildings using nonlinear dynamic analysis. The BC guidelines were written to be consistent with the PEER TBI guideline and/or the LATBSDC guidelines where ever possible, while fully complying with the Canadian design requirements in NBC and CSA A23.3, which have some significant differences with the two US Codes mentioned above. The BC guidelines do not repeat information contained in the PEER TBI guidelines and LATBSDC guidelines, and so the BC guidelines are meant to be used in conjunction with either of these US guidelines.

In this paper, the authors of the BC guidelines (Section 3.4.8 of the EGBC practice guideline on tall concrete building design) provide a high-level summary of the guidelines, explain the rationale behind the development of the guidelines, and provide a discussion on some of the differences between the BC guidelines and the LATBSDC guidelines.

SEISMIC DESIGN OF SHEAR WALLS USING CSA A23.3 CLAUSE 21

In order to understand the BC guidelines, it is necessary to have a high-level understanding of the CSA A23.3 prescriptive procedures for the seismic design of shear walls. As mentioned above, CSA A23.3 Clause 21 contains displacement-based procedures. The average effective flexural rigidity $E_c I_e$ of wall piers and the effective axial rigidity $E_c A_e$ of coupled wall piers that are used to estimate the top wall displacement are a function of the ratio of elastic bending moment demand (determined by modal response spectrum analysis) to nominal flexural resistance of the wall. The expression given in CSA A23.3 was developed [4] to give a similar top wall displacement from modal response spectrum analysis as is obtained from NLTHA.

The top wall displacement is the engineering demand parameter used to estimate the inelastic deformation demands such as the rotation demand on wall piers. A portion of the total displacement is attributed to elastic deformation of the walls. Based on the results of nonlinear time history analysis, the elastic portion of the wall displacement is again estimated from the ratio of elastic bending moment demand to nominal flexural resistance of the wall. The inelastic rotation is equal to the inelastic displacement divided by the height above the centre of rotation at the mid-height of the plastic hinge. The inelastic rotation capacity is calculated from the curvature capacity associated with a maximum concrete compression strain of 0.0035, an upper-bound estimate of the elastic (yield) curvature (inelastic curvature is the difference between total curvature and elastic curvature), and a lower-bound estimate of the plastic hinge length.

An upper-bound estimate of the inelastic rotational demand on coupling beams is determined from the global drift ratio and the ratio of horizontal distance between centroids of walls on either side of coupling beam to the clear span of the coupling beam. This estimate ignores the differential "floor slope" (due to axial strain of wall piers on either side of coupling beams) over the height of the building and thus predicts a constant coupling beam rotation over the building height. The inelastic rotation capacity of coupling beams with diagonal reinforcement is taken as 0.04 based on coupling beam test data available prior to the 2014 edition of CSA A23.3 when this provision was first introduced. As discussed later, these limits require updating.

The inelastic rotation demand in wall piers is used to reduce the concrete contribution to shear resistance V_c for the wall and the maximum shear resistance V_{rmax} as limited by the crushing strength of concrete subjected to shear (biaxial strains). The compression stress angle θ used to calculate the horizontal wall steel contribution to shear resistance V_s depends on the axial compression stress level in the wall. [5]

The shear demand is adjusted to account for flexural overstrength of the walls and then further increased to account for inelastic effects of higher modes. To reduce the likelihood of a shear failure, the shear force demands determined from a modal response spectrum analysis are multiplied by the ratio of probable bending moment resistance to factored bending moment calculated at the critical zone where flexural yielding will first occur. This is often a multiplier of 2.0 or more. In addition, to account for the fact that only the first mode shears are limited by the formation of a plastic hinge at the base of a wall (higher mode shears are less influenced by the formation of a plastic hinge at the base), the shear force demands are further increased depending on the ratio of elastic bending moment demand to nominal flexural resistance of the wall. The maximum additional increase is limited to a multiplier of 1.5. The rationale for not increasing the shear demands further to match the single largest shear force pulse that would be determined by a nonlinear time history analysis includes: (i) the higher mode shear force pulse last for a very short time and the second largest shear force pulse is often 20% less, (ii) the maximum shear force usually occurs when the inelastic rotation demands in the wall are lower (i.e., during elastic cycles), (iii) a concrete shear wall has considerable shear ductility, as well as other factors. [6]

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 hinges in the tower walls usually happen above this level. The system of walls and diaphragms that resist the overturning moment and shear forces below the plastic hinge is indeterminate. CSA A23.3 permits multiple static analyses with varying stiffnesses [7] to be used to analyze the system in order to determine the design forces for the shear wall below the plastic hinge and the design forces for the foundation accounting for foundation movement due to flexibility of soil and uplift of the footing [8] as required by NBC.

CSA A23.3 has extensive requirements to ensure the gravity-load resisting frame members are able to tolerate the building displacements either by being flexible enough to tolerate elastic deformations or sufficiently ductile to tolerate inelastic deformation while continuing to support the gravity loads. Clause 21.11 of CSA A23.3 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. [9]

BC Guidelines for Design of Tall Concrete Buildings

The BC guidelines for the design of tall concrete buildings (EGBC professional practice guideline *Structural Engineering Services for Tall Concrete Building Projects*) [3] covers three main topics as summarized below.

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.

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.

Design for earthquake ground motions provides a detailed explanation of the procedures briefly summarized above. It 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. The final section (3.4.8) is on evaluating life safety performance using nonlinear dynamic analysis and is herein referred to as the BC Guidelines for seismic design of tall concrete buildings using nonlinear dynamic analysis.

BC GUIDELINES FOR DESIGN USING NONLINEAR ANALYSIS – MODELLING CONSIDERATIONS

Similar to the other guidelines, the BC guidelines include a general requirement that the structural model incorporate realistic estimates of stiffness and damping considering the expected levels of excitation and damage. The characteristics of the nonlinear 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 nonlinear range.

The nonlinear 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.

Most floor diaphragms can be modelled as rigid; however, 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 at lower levels where multiple elements of SFRS share a common diaphragm.

The horizontal mass must be accurately distributed in plan to accurately account for torsional inertial effects. The mass below-grade structure is normally not included. LATBSDC guidelines are referenced for additional guidance. 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. Accounting for accidental torsion by shifting the centre of horizontal mass in four different directions significantly increases the number of nonlinear analyses that need to be done. In order to reduce the number of nonlinear analyses, both the LATBSDC guidelines and the PEER TBI guidelines suggest that accidental eccentricity need not be added to the nonlinear model when a building has low torsional sensitivity. The BC guidelines refers to these other guidelines for how to deal with accidental torsion.

Vertical mass must be included in the model whenever the structural configuration is such that the horizontal motions in the building can induce vertical motions, such as with sloped columns. Further, vertical ground motions must be included in the analysis when: (i) a significant discontinuity exists in a vertical-load-carrying element, such as a gravity-load column supported on a transfer slab or girder; (ii) a gravity-induced lateral demand (GILD) irregularity exists in the building; or (iii) a sloped-column irregularity exists (vertical member, inclined more than 2 degrees from the vertical, supports a portion of the weight of the building in axial compression).

In nonlinear analysis, the damping provided by hysteretic energy dissipation of structural members is modelled explicitly. An additional amount of equivalent viscous damping is usually included in the model to account for the inherent damping of the structure that is not associated with nonlinear elements. The BC guidelines follow the other guidelines for the reduced damping in tall concrete buildings.

The superposition of design forces cannot be used with nonlinear analysis results. Therefore, the gravity loads dead (D), live (L) and snow (S) that occur simultaneously with the earthquake forces (E) must be applied to the model of the building for the nonlinear earthquake analysis. According to NBC, the two load combinations that need to be considered are: 1.0D + 1.0E + 0.5L + 0.25S; and 1.0D + 1.0E. For tall concrete buildings, 0.5L + 0.25S is usually a small addition, and thus, it is usually not necessary to repeat the nonlinear analysis with the second load combination. The maximum gravity loads must be included in the analysis in order to account for the geometric nonlinearity due to second-order bending moments (P-Delta), and these loads must be accurately distributed to capture the influence of both building translation and twist. The BC guidelines recommend accounting for the reduced gravity loads when calculating the flexural resistance of wall piers.

Component monotonic backbone curves and cyclic deterioration characteristics are to be established from physical test data, or from analytical approaches that have been benchmarked to physical test data. 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. Specific guidance is provided for wall piers, coupling beams, transfer slabs and slab-column connections.

BC GUIDELINES – REQUIRED NUMBER OF ANALYSES AND COMPONENT STRENGTHS

Consistent with the PEER TBI guidelines and the LATBSDC guidelines, seismic demands (actions) are classified as either deformation-controlled demands or force-controlled demands. Deformation-controlled demands include deformations and forces associated with a ductile nonlinear response under reversed cyclic loading. For concrete buildings, this includes: inelastic rotation of wall piers; 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 nonlinear 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 nonlinear element.

The Structural Commentaries to NBC 2015 states that two full sets of nonlinear analyses need to be done with different component strengths to determine separately the deformation-controlled demands and the force-controlled demands. As per NBC 2015, lower-bound component strengths need to be used when determining deformation-controlled demands, while upper-bound component strengths need to be used to determine force-controlled actions. The recommended lower-bound strength 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.

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 nonlinear 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 nonlinear analysis. These analyses need to be repeated four times if accidental torsion is included. Thus, the BC guidelines recommend that one set of component strengths be used, calculated using concrete and reinforcement material strengths of $1.2f_c'$ and $1.2f_y$.

The recommendation for component strengths was based on the following. The typical grade of reinforcement in BC is 400W, meeting CSA G30.18, 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 usually have an actual yield strength of about 460 MPa and minimum ultimate strength of 540 MPa.

BC GUIDELINES – SEISMIC HAZARD AND GROUND MOTION TIME HISTORIES

The BC guidelines provide guidance for the selection and scaling of time histories to be used in the nonlinear time history analysis of tall concrete buildings. It generally follows the procedures given in the Appendices of NBC 2015 and 2020 Commentary J; with modifications based on the consensus of recommended practice in BC.

Design Spectrum

The 6th Generation Seismic Hazard Model of Canada [10], which was developed for the seismic design values in NBC 2020, should be used to determine seismic hazard at the site. For 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 in accordance with NBCC 2020, as linear interpolation of acceleration at widely spaced periods (2, 5, 10 s) results in a highly distorted displacement spectrum.

Period Range

To select and scale time histories, 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. T_{min} is taken as the smaller of 0.15 T_1 or $T_{90\%}$, where T_1 is 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 1.5s. If the nonlinear analysis results are not used for design of subterranean elements, $T_{90\%}$ can include only the mass of superstructure.

Selection of appropriate ground motions is limited to the ground motions that have similar geophysical parameters (site conditions, magnitudes) and distances to the earthquake hazards that dominate the Uniform Hazard Spectrum (UHS). Southwestern BC has a unique subduction zone setting that includes earthquakes from three sources: crustal events, which occur along shallow faults in the Earth's crust; Inslab events, which occur deep within subducting tectonic plates; and subduction interface events, which are caused by slip between tectonic plates.

Appendix X of Commentary J of NBC 2015 recommends a minimum of one scenario-specific period range (T_{RS}) for each tectonic source contributing to the hazard in Southwestern BC (i.e., three T_{RS} comprised of one each: $T_{RS\ Crustal}$, $T_{RS\ Subcrustal}$, and $T_{RS\ Interface}$). Based on the consensus of experts consulted in the development of the 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 of design UHS. The second specific period range $T_{RS\ long}$ must be developed over the long period range for subduction interface motions. This is consistent with upcoming Appendix X of Commentary J of NBC 2020 which allows to combine the shallow crustal and in-slab earthquakes if they cover the same period range.

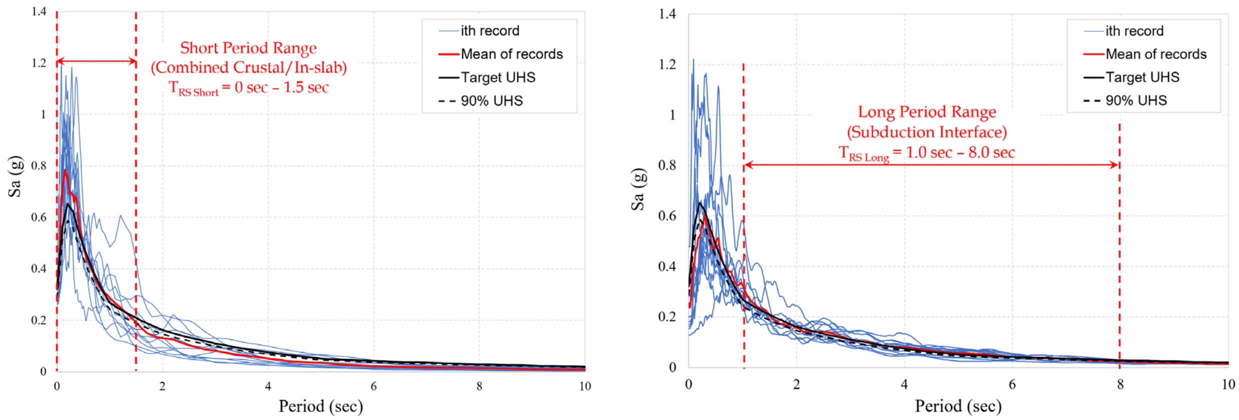
Method A and B of Appendix X of Commentary J of NBC 2015 and 2020 can be used to develop the target response spectra, $S_T(T)$, for the horizontal component of ground motions. Recent hazard models developed for NBC 2020 provide data only up to a period of 10 s and therefore caution should be used when the fundamental lateral period of the building, T_1, is greater 5 s. Due to this limitation, the upper bound range of period for scaling the ground motions 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.

Number of Ground Motions

As mentioned above, two sets of 11 ground motions are to be used (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 Commentary to NBC 2015 recommends not less than 11 ground motions over three scenario period ranges (33 motions in total), it permits as few as 3 sets of 5 ground motions (15 motions in total). The upcoming Appendix X of Commentary J of NBC 2020 does not permit less than 11 ground motions for each seismic source. However, it makes an exception for Southwestern BC to combine crustal and in-slab records if they cover same period range and have a suite of not less than 11 records provided the records are peer reviewed.

Based on these guidelines and the upcoming Appendix X of Commentary J of NBC 2020, it is recommended to use the 22 motions as described above in Southwestern BC. An example for 2 sets of 11 ground motions selected and scaled to period range of T_R = [0 – 8 s] for a site in Vancouver is shown in Figure 1. 11 crustal and inslab records are scaled to $T_{RS\ short}$ = [0 – 1.5 s] (Figure 1.a) and 11 subduction interface records are scaled to $T_{RS\ long}$ = [0 – 1.5 s] (Figure 1.b).



(a) 11 crustal and in-slab records *(b) 11 interface subduction records*

Figure 1. Selected and scaled ground motions for a site in Vancouver (a) 11 crustal and in-slab records and (b) 11 subduction interface records

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. 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 must not fall below 110% of the target spectrum at every period of T_{RS}.

Vertical Spectrum and Ground Motions

When accounting for vertical response, as required by these guidelines, the vertical component of the ground motions should be included in the analysis and should be scaled by the same factor as the corresponding horizontal ground motion records. The compatibility of the scaled vertical component spectra with the target vertical spectra should be checked and 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 [11]. It is not recommended to use the commonly applied factor of 2/3 to the horizontal-component target spectrum for determining the vertical-component target spectrum

BC GUIDELINES – 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: (i) deformation demands on deformation-controlled actions or elements are within the limits specified in CSA A23.3, (ii) strength demands on force-controlled actions or elements are smaller than the factored strengths calculated in accordance with CSA A23.3, (iii) a maximum of one unacceptable response occurs for the suite of 11 ground motions, (iv) peak transient drifts and residual drifts are within acceptable levels. 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%.

The demand on the structure is determined from nonlinear analysis using a model of the building having component strengths calculated using the "expected" concrete and reinforcement material strengths of $1.2f_c'$ and $1.2f_y$; however, the deformation capacities and strength capacities are calculated using the procedures in CSA A23.3, with factored material strengths $0.65f_c'$ and $0.85f_y$.

As described above, two suites of 11 ground motions selected and scaled over scenario-specific period ranges are used to conduct the nonlinear 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.

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. 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%. 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.

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.

Deformation Demands

Core walls are designed to dissipate energy by flexural yielding of the wall piers and flexural/shear yielding of the coupling beams. Core walls are designed for flexural yielding to occur within the "plastic hinge regions," which contains special detailing. 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. The BC guidelines permit two different procedures for evaluating the deformation demands on wall piers. In the first, the inelastic deformation demands on wall piers are investigated in terms of wall rotations using the CSA A23.3 procedures described above. The second procedure that is permitted is to use compression and tension strain demands directly. 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. The maximum compression strains determined directly from the nonlinear 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 nonlinear 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 nonlinear 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 the plastic hinge regions, the wall has less special seismic detailing, and therefore lower strain limits are appropriate. The maximum compression strains in concrete 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 nonlinear 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.

Two different procedures are also permitted for evaluating the deformation demands on coupling beams. The first is the limit on inelastic rotation given in CSA A23.3 and described above (e.g., 0.04 for coupling beams with diagonal reinforcement). In the second procedure, the total rotational demands on coupling beams are limited to 0.06 for coupling beams with diagonal reinforcement; or 0.03 for coupling beams without diagonal reinforcement.

Force Demands

The shear force demands on wall piers are a very important part of evaluating life safety performance of highrise concrete shear wall buildings. This is discussed in detail in the next section.

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 nonlinear 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 nonlinear elements allowing the deformation demands (bending moment) and force-controlled demands (shear and axial force) to be assessed.

Force-controlled actions are further classified into different categories of criticality. 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 nonlinear 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.

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.

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 nonlinear dynamic analysis.

SHEAR FORCE DEMANDS ON WALL PIERS

Ensuring the shear force demands on wall piers will not result in shear failure of the wall piers is a very important part of ensuring life safety performance of highrise concrete shear wall buildings. According to the BC guidelines, the mean shear force demand determined from nonlinear 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.

The BC guidelines recommends that the mean shear force demand determined from nonlinear dynamic analysis not be increased by the 1.5 or 1.3 factor used by the LABSDC guidelines and PEER TBI guidelines 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. 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. [5]

Comparison of shear designs using the LATBSDC guidelines and the BC guidelines/CSA A23.3 Clause 21.5.9 has shown that the resulting designs for a variety of buildings are very similar even though the procedures are very different. The LATBSDC guidelines use expected material strengths $1.3f_c'$ and $1.17f_y$, use the full length of wall as the shear depth (length over which the shear stresses can be assumed uniform), but uses a stress field angle θ = 45 deg. to determine the steel contribution and compares the shear resistance to 1.5 times the mean shear demand. The BC guidelines procedure following CSA A23.3 Clause 21.5.9 uses the factored material strengths $0.65f_c'$ and $0.85f_y$, uses 80% of the wall length as the shear depth, but uses a stress field angle θ = 35 deg. to determine the steel contribution outside the plastic hinge region and according to the BC guidelines, you compare that factored shear resistance to 1.0 times the mean value determined from NLTHA. CSA A23.3 shear design provisions make the maximum shear resistance (to avoid diagonal crushing of concrete) proportional to the concrete compression strength. As a result, much higher shear stresses are permitted by CSA A23.3 outside the plastic hinge region.

Figures 2(a) and 3(a) compare the results from the BC guidelines/CSA A23.3 Clause 21.5.9 with LATBSDC guidelines for outside the plastic hinge region in the wall. Figure 2(a) shows the maximum shear stress (to avoid diagonal crushing) and the concrete contribution for a range of specified concrete compression strengths. Figure 3(a) shows how the resulting quantity of horizontal reinforcement in the wall is very similar outside the plastic hinge region.

Within the plastic hinge region, the LATBSDC procedure reduces the shear strength of the wall by 33% when the maximum steel strain in the wall is greater than or equal to 0.03. As the steel increases from 0.01 to 0.03, the shear strength is reduced linearly from 100% to 67%. Within the plastic hinge region, CSA A23.3 reduces the concrete contribution and reduces the maximum shear stress as a function of the inelastic rotation. The minimum values are reached when the inelastic rotation is greater than or equal to 0.015. The concrete contribution and maximum shear stress are reduced linearly as the inelastic rotation increases from 0.005 to 0.015. As the compression strain depth in a typical highrise core wall pier is a small portion of the wall length (e.g., 15% of the wall length), the maximum tension strain is approximately twice the inelastic rotation if the plastic hinge length in the wall is assumed to be half the wall length as is the case in CSA A23.3. Thus the linear reductions in shear resistance occur in the LATBSDC and CSA A23.3 procedures at similar deformation levels. Figure 2(b) compares the maximum shear stress limits when the maximum reductions are applied, and Figure 3(b) compares the required quantity of horizontal reinforcement in a wall with f'_c = 50 MPa when the maximum reductions are applied. It is remarkable how similar the resulting designs are from these two very different procedures.

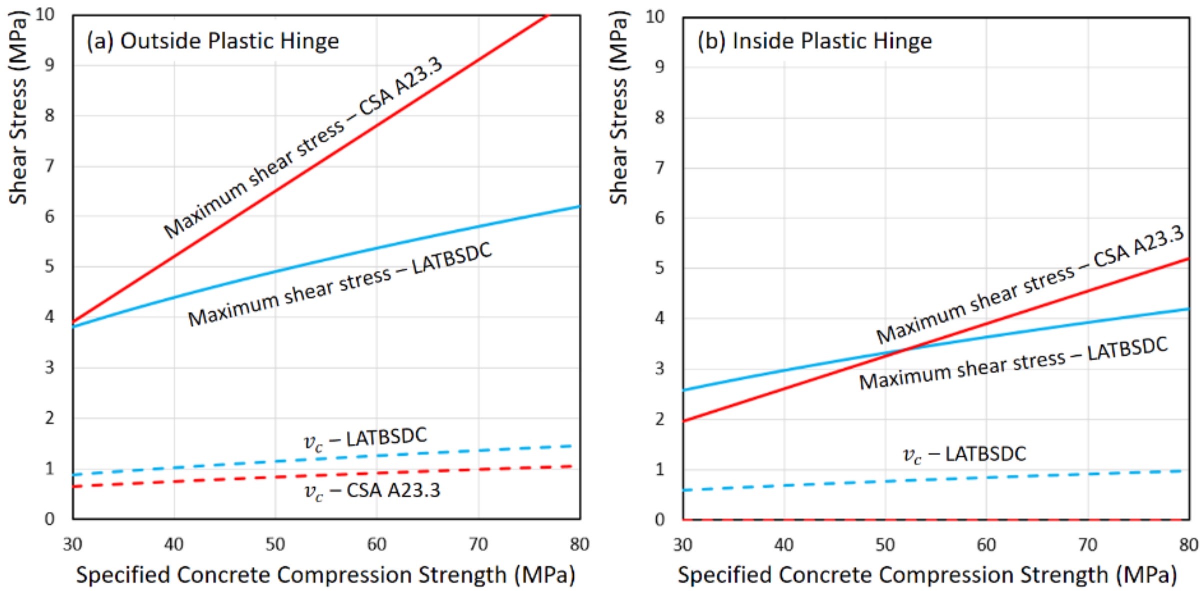


Figure 2. Comparison of maximum shear stress resistance (solid lines) and concrete contribution (dashed lines) calculated using BC guidelines/CSA A23.3 and LATBSDC guidelines: (a) outside the plastic hinge region in wall, and (b) inside the plastic hinge region.

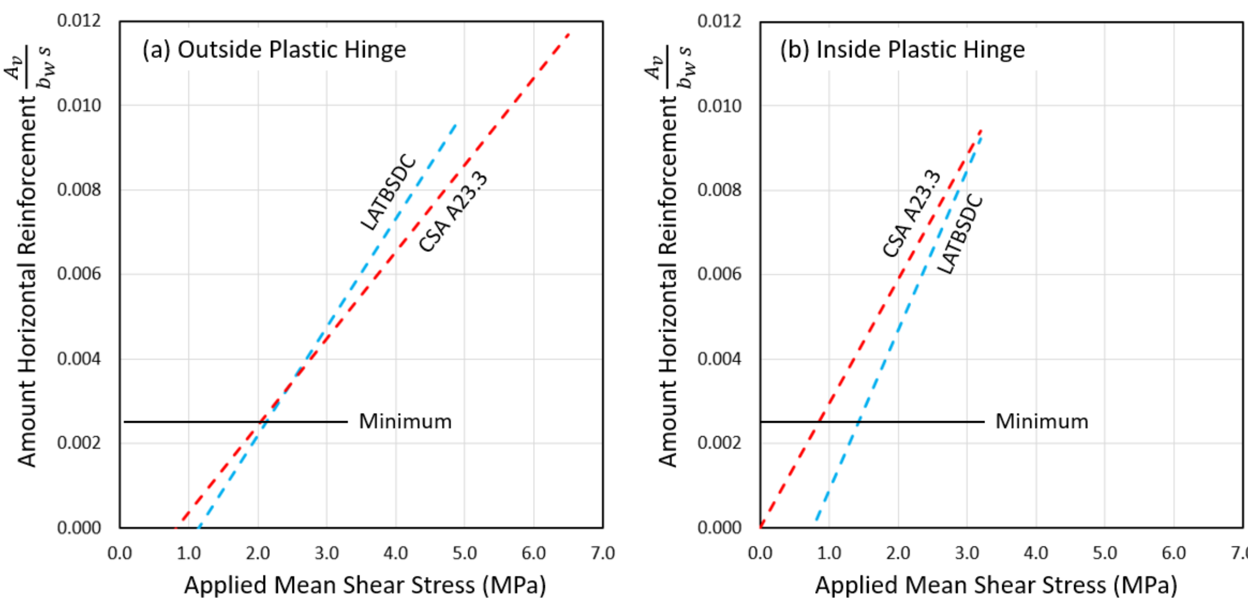


Figure 3. Comparison of required amount of horizontal wall reinforcement in a wall with f'_c = 50 MPa calculated using CSA A23.3 and LATBSDC guidelines: (a)outside plastic hinge region in wall, and (b) inside plastic hinge region.

TOWARDS AN UPDATE TO THE BC GUIDELINES

Serviceability Evaluation

The 2020 edition of NBC introduced additional performance requirements for normal importance category buildings with a height above grade more than 30 m in Seismic Category SC4, which includes the seismically active regions of BC. In these buildings, the structural framing elements not considered part of the SFRS are required to behave elastically when subjected to ground motions having a 10% probability of exceedance in 50 years.

The 2024 edition of NBC is expected to provide additional guidance on the requirement, including: (i) the SFRS and structural elements not part of the SFRS may be used to resist earthquake loads and deflections; (ii) the structural modeling shall include stiffnesses that are appropriate for structural elements that remain elastic, accounting for the expected level of cracking; (iii) the analysis of buildings with Type 9 (GILD) irregularity or Type 10 (sloped-column) irregularity shall account for the vertical response of the building mass and shall include vertical ground motions, and; (iv) the increased displacements of the structure resulting from foundation movement need not be considered.

Designers require additional guidance to conduct the required serviceability evaluation of highrise concrete shear wall buildings. The expression given in CSA A23.3 for the average effective flexural rigidity $E_c I_e$ of wall piers and the effective axial rigidity $E_c A_e$ of coupled wall piers, which is a function of the ratio of elastic bending moment demand to nominal flexural resistance of the wall, can be used to estimate the increased stiffness of shear walls that have lower levels of damage. However, specific recommendations are required for the effective axial, flexural and shear rigidity for the structural framing elements that are required to remain elastic.

Guidance is also required for a suitable acceptance criteria for “elastic behaviour.” For structural concrete elements that are predominantly subjected to flexure, a simple strength check can be used. Structural concrete elements that are expected to develop significant diagonal cracking will require special crack control reinforcement to ensure elastic behavior (large diagonal cracks cause residual deformations). An orthogonal grid of reinforcing bars near each face of the member with a minimum ratio of reinforcement area to gross concrete area not less than 0.002, and a maximum spacing of 300 mm for this reinforcement would seem to be a suitable recommendation.

Update Rotational Capacity of Coupling Beams

As mentioned previously, the inelastic rotation capacity of coupling beams with diagonal reinforcement was introduced into CSA A23.3 in 2014 when there were significantly fewer coupling beam tests available. A recent review of all available test data [13] has demonstrated the slender diagonally-reinforced coupling beams can tolerate larger inelastic rotations than the current 0.04 limit, while squat (deep) coupling beams have a lower inelastic rotation capacity. As these rotational limits are often critical in the design of a highrise core, it will be important to update these limits.

Guidelines for the Design of Outriggers

The geometry of the core in a shear wall building must meet both functional and structural requirements. Over the past forty years that cores have been commonly used in BC, the functional requirements have aligned well with the structural requirements. More recently, due to such factors as improved elevator technology, the cores have become increasingly slender, so much so that the performance of the taller buildings are reduced under wind and seismic demands. One solution is to add outriggers supported on columns to reduce the wind and seismic demands on the core. Currently, no guidelines exist in Canada for the design of such outriggers, so it would be desirable to add such information to the next edition of the BC guidelines.

To demonstrate the potential impact of outriggers, a study was conducted on a 52-story building with six levels of podiums and two levels below grade. The original (base) building has only a centrally located core. Two different types of outriggers were used – three-storey high steel outrigger trusses with buckling restrained braces (BRBs), and one-storey high concrete outriggers. Four steel truss outriggers were used on each side of the building supported on four columns (the BRBs have a capacity of 1500 kips each), while two concrete outriggers supported by columns were used on each side of the building. The steel truss outriggers were located at Levels 17 to 20 (approx. 1/3 of building height) or Levels 32 to 35 (approx. at 2/3 of building height), while the concrete outriggers were located at Levels 20 or 35.

It is interesting to compare the modal periods for the five different buildings and to see how the two different types of outriggers and two different outrigger locations influence the different modal periods. The original building with no outriggers has a fundamental lateral period T_1 = 6.66 s. The outriggers with the least effect on the fundamental lateral period is the steel BRB outriggers located at approximately one-third of the building height (T_1 = 5.24 s). Concrete outriggers at the same location reduced T_1 to 4.91 s. When the steel BRB outriggers are moved to two-thirds the building height, T_1 reduces to 4.89 s, while when the concrete outriggers are located at two-thirds the height, T_1 = 4.81 s. The influence of the outriggers on all the other modal periods is much smaller.

Figure 4 shows how the different outriggers influenced the shear force and bending moment due to ULS wind loading. The outriggers cause a significant reduction in the bending moment that must be resisted by the core. The sudden reduction in bending moment at the outrigger location causes a large shear force reversal, particularly for the building with the concrete outriggers. This very large shear force is an important design consideration for the core wall design. As the maximum shear force is very localized, the design for this shear must account for exactly how the forces are applied to the core. The strut-and-tie procedure with CSA A23. is the appropriate procedure to accomplish that.

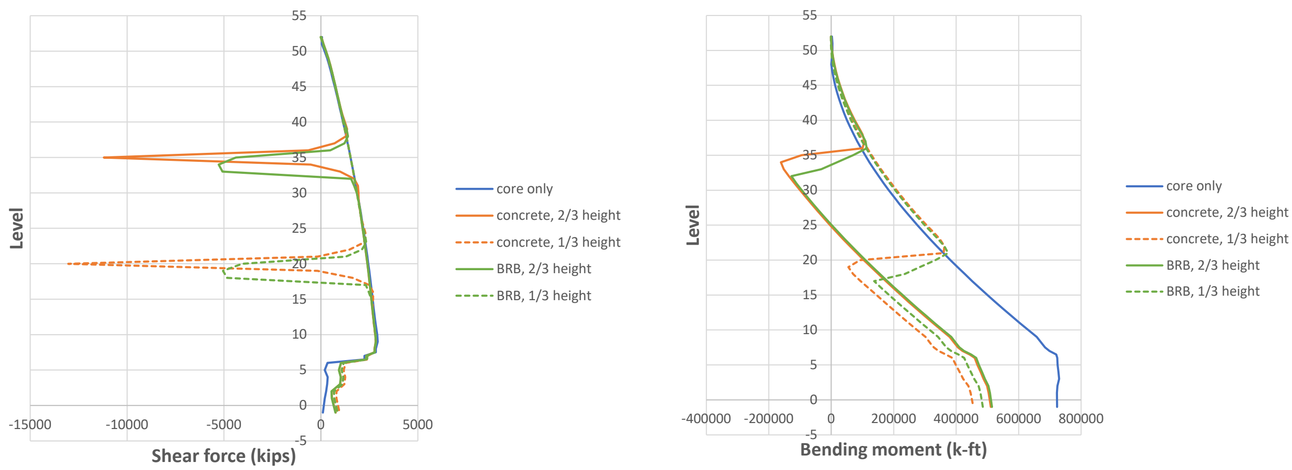


Figure 4. Comparison of shear force demands (left) and bending moment demands (right) due to ULS wind for a 52-storey building with five different arrangements of outriggers.

NLTHA was used to study the seismic performance of the five buildings. Figure 5 presents the mean interstorey drifts for three different arrangements of outriggers – no outriggers, steel truss outriggers with BRBs at 2/3 the height and concrete outriggers at 2/3 the height. The nonlinear analysis shows that the additional damping from the BRBs significantly reduced the interstorey drifts compared to the concrete outriggers.

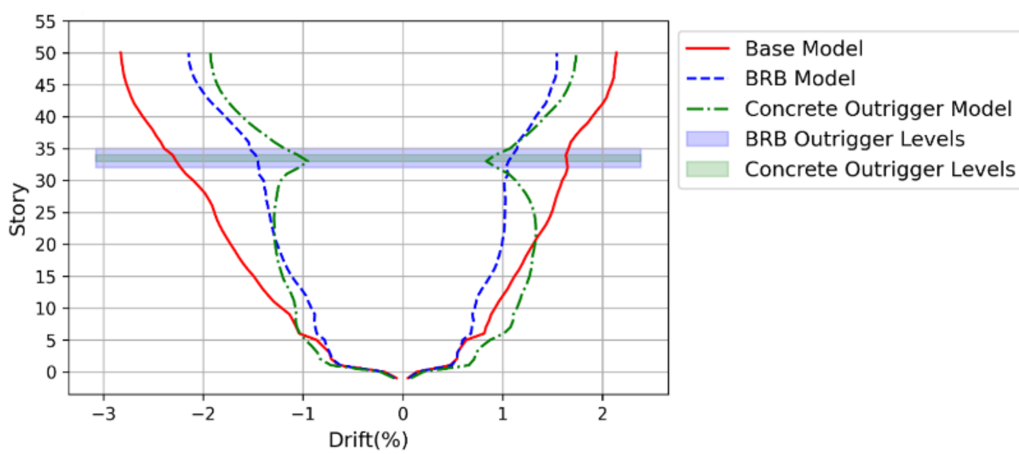


Figure 5. Comparison of mean interstorey drifts determined from NLTHA on a 52-storey building with three different arrangements of outriggers.

Performance-Based Wind Design

For tall concrete buildings, the wind design requirements often control a significant part of the design, such as the required strength of the coupling beams, and the quantity of vertical in the wall piers at the base. The CSA capacity design requirements means that the entire strength of the core and the foundation need to be increased proportionally in order to ensure the weak link is the ductile deformation controlled elements (e.g., the coupling beams). If the coupling beam shear force demands could be redistributed over a few stories, the required overstrength of the core could be significantly reduced. Unfortunately, this is not permitted according to current design procedures for wind.

The performance-based design for wind (PBDW) is a new approach that would allow redistribution of coupling beam forces as well as other limited inelasticity in certain deformation-controlled elements. PBDW requires NLTHA for wind demands with appropriate nonlinear force-deformation relationship of deformation-controlled elements. The 2025 edition of ACI Code 318 Building Code requirements is expected to have guidelines for PBDW of concrete buildings. A realistic goal for the next edition of the BC guidelines is to include procedures for allowing a limited redistribution of coupling beam forces based on a PBDW approach.

CONCLUSIONS

As building architecture is becoming more unique and less like the buildings that prescriptive procedures were developed for, the need for nonlinear time history analysis (NLTHA) to confirm the life safety performance of buildings has increased. While excellent guidelines for the seismic design of buildings using NLTHA are available from the United States, these guidelines do not result in designs that are consistent with the Canadian prescriptive design requirements in NBC and CSA A23.3. Thus, specific guidelines were developed by the authors for use in BC, and recently published as part of the EGBC professional practice guideline *Structural Engineering Services for Tall Concrete Building Projects*.

This paper provided a high-level summary of the guidelines, explained the rationale behind the development of the guidelines, and provided a discussion on some of the differences between the BC guidelines and the LATBSDC guidelines. A detailed comparison of the shear design requirements for concrete walls shows that while the procedures are very different, the resulting designs are very similar. Information was also presented about expected changes and additions to the next edition of the BC guidelines, including the addition of serviceability evaluation of highrise concrete buildings consistent with the latest editions of NBC, the addition of design guidelines for outriggers, and the use of NLTHA of wind demands to permit the redistribution of wind force demands on coupling beams.

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