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**Impact of soil-structure interaction on seismic design of steel frames  
foundations**

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Département des génies civil, géologique et des mines

Mémoire présenté en vue de l'obtention du diplôme de *Maîtrise ès sciences appliquées*

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présenté par **Angel REYES FERNANDEZ**

en vue de l'obtention du diplôme de *Maîtrise ès sciences appliquées*

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## DEDICATION

*In memoriam of Professor Salvador Mora Moran.*

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## RÉSUMÉ

Afin d'assurer une réponse appropriée d'un bâtiment lors d'un tremblement de terre, les normes et standards canadiens établissent les exigences minimales pour la conception parasismique. L'approche de conception par capacité est mise en place comme méthode principale pour assurer une résistance, une rigidité et une stabilité suffisantes des structures en réponse aux charges sismiques. La conception par capacité est basée sur l'idée que les déformations inélastiques qui se développent dans les éléments spéciaux du système de reprise des charges sismiques (SFRS) connus comme des fusibles dissipent l'énergie introduite par les seimes. Le dimensionnement du reste des éléments SFRS doit se faire de façon à maintenir la capacité maximale de ces fusibles. Cette approche peut conduire à de grandes dimensions d'éléments sur le chemin de descente de la charge sismique, du sommet jusqu'à la fondation. À la fin, les fondations sont dimensionnées pour résister la capacité probable du système de reprise des charges sismiques (fondation « capacity-protected ») ou d'une partie de celui-ci (fondation « not-capacity protected »).

Les éditions actuelles des codes de construction canadiennes font une distinction explicite entre les fondations qui permettent un comportement basculant lors d'un tremblement de terre et celles qui sont restreintes aux mouvements de rotation. Cette distinction joue désormais un rôle important lors de la conception du SFRS si les fondations basculantes sont utilisées. Les fondations basculantes entraînent des déplacements inter-étage supplémentaires dans la super-structure qui doivent être pris en compte lors de l'estimation des déflexions latérales. Pour contrôler les déplacements latéraux de la superstructure, la taille de la fondation est le plus souvent augmentée ce qui peut conduire aux très grandes fondations qui peuvent des fois mettre en question le choix d'un système en acier pour la reprise des charges sismiques.

Étant donné que la grande majorité des analyses structurales effectuées par les ingénieurs emploie des modèles numériques à base fixe, la rotation attendue de la semelle est estimée par des procédures empiriques proposées par le standard de conception des structures en béton armé, CSA A23.3. Pour les murs de refends en béton armée, ces procédures ont montré une bonne concordance les murs de refends en béton armée en comparaison avec les résultats obtenus par les autres méthodes analytiques plus sophistiquées. Vu que la nature de plastification des cadres contreventés en acier est différente, la question se pose si ces méthodes d'estimation sont appropriées pour ce type de système.

Les analyses tenant compte de l'interaction sol-structure (ISS), ont démontré que , dans certains cas, une réponse sismique adéquate des cadres contreventés est atteinte, même si les dispositions relatives à la rotation imposées par le A23.3 sont ignorées.

Pour valider l'impact d'ISS sur la conception des fondations des cadres concentriques en acier, les bâtiments prototypes de 3 et 6 étages ont été conçus pour Vancouver et Montréal sur la catégorie de site C et E et leur réponse sismique a été étudié en utilisant les analyses non-linéaires effectués avec OpenSees. Les différents aspects de la réponse inélastique des cadres en acier contreventés concentrique, tels que la plastification en traction, le flambement en compression, l'influence des assemblages et l'effet des extensions rigides, ont été explicitement introduits dans les modèles numériques. Des enregistrements sismiques appropriés pour l'est et l'ouest du Canada ont été utilisés pour effectuer des analyses temporelles non linéaires.

L'ISS a été implémenté en utilisant la méthode *Beam on Nonlinear Winkler Foundation* (BNWF) dans les modèles. Cette méthode est appliquée par une modélisation directe de la fondation sous la forme d'une poutre élastique, maillée sur sa longueur et reliée à chaque discrétisation à un nœud fixe par un ressort non linéaire calibré pour refléter le comportement inélastique du sol. Cette technique permet de simuler la réponse en compression du sol, préalablement calibrée, et d'éviter toute résistance à la traction permettant à la semelle de se soulever. Un amortissement supplémentaire lié au ISS peut également être introduit.

Les résultats de simulation des scénarios étudiés lorsque ISS est considéré pour les sites avec un sol rigide, comme le site de catégorie C, montrent que la réponse basculante peut en fait servir de mécanisme de dissipation d'énergie supplémentaire. Le basculement peut limiter l'entrée de l'énergie sismique dans le SFRS, ce qui entraîne un moment de renversement dans la fondation inférieur au moment probable du SFRS. Les tassements, les soulèvements et les rotations mesurés ne dépassent pas les valeurs recommandées pour une réponse adéquate. Les déplacements inter-étage et les efforts dans les ressorts sont dans une plage acceptable pour assurer une bonne performance.

Les résultats observés dans les sols mous, (site catégorie E), suggèrent une condition possiblement problématique pour le système sol-fondation-structure. Les dispositions actuelles du code ont été jugées adéquates pour tels cas afin d'assurer une réponse acceptable. Si les dispositions relatives à

la rotation ou les dispositions relatives à la conception de la capacité sont ignorées, des tassements inacceptables ont été observés.

Une dimension de fondation plus petite peut être obtenue lorsque les conditions du site sont favorables, ce qui réduit les coûts de construction et favorise les SFRS basés sur des cadres contreventés en acier. Par contre, précaution est suggérée lorsque les conditions du site sont moins favorables.

## ABSTRACT

In order to ensure an adequate response during an earthquake, National Building code of Canada (NBC) and standards for design of steel and concrete structures ( S16 and A23.3) establish the minimum provisions for earthquake resistant design. Capacity design approach is implemented as primary method to provide sufficient strength, stiffness and stability to the structure in response to seismic loads. Capacity design approach is based on the idea that the inelastic deformation of seismic force resisting system (SFRS) are confined to the so-called fuse elements, who dissipate earthquake energy, while the rest of the SFRS elements is designed to sustain the maximum capacity of these fuses. This approach can lead to large element dimensions along the seismic load path, from the roof of the building down to the foundation. Ultimately, foundations are dimensioned to resist the probable capacity of the SFRS (*capacity-protected* foundation) or a portion of this capacity (*not-capacity protected* foundation).

Recent editions of the Canadian construction codes and standards make an explicit distinction between foundations that allow a rocking behavior during an earthquake and those that are restrained to rotational movements. This distinction is now playing an active role during the design of the SFRS if rocking foundations are used. Rotations of rocking foundations cause an additional inter-storey drift of the super-structure that must be considered when the lateral deflections are estimated. To control the deformation increase, oftentimes the footing sizes are increased, resulting in a significant augmentation of footing size if lateral deflection limits are not satisfied. The dimensions and the associated cost of such foundation can be so important that the choice of steel SFRS can be put in question

Since the vast majority of the structural analyses made in practice by structural engineers considers that the building is fixed at its base, the expected footing rotation is most often estimated by empirical procedures proposed by CSA A23.3. These procedures have shown a good match for reinforced concrete shear walls when compared with the results of more sophisticated analytical methods. Considering a very different yielding mechanism typical for concrete shear walls and steel concentrically braced frames, one can question if the approximate procedures are well suited for steel systems. Examples of the previous analyses in which *soil structure interaction* (SSI) was implemented, have shown in some cases an adequate seismic response of braced frame when rotation-related inter-storey drift requirements from A23.3 were not fully respected.

To validate Canadian seismic design provision for foundation design of steel concentrically braced frames, 3- and 6-storey prototype buildings with moderately ductile concentrically braced frames with X bracing were designed for Vancouver and Montreal on Class C and Class E sites. Foundation design included capacity-protected and not-capacity protected options. The numerical models were developed in OpenSees. Inelastic frame response including tensile yielding, compressive flexural buckling, gusset plate influence and rigid extension effect was explicitly introduced into the numerical models. Ground motions record for the East and the West of Canada, compatible with design spectra for studied locations were selected and employed to perform time-history non-linear analyses.

SSI were implemented by using the Beam on Nonlinear Winkler Foundation (BNWF) method. In this method the foundation is modeled as an elastic beam, meshed along its length and connected at each discretization to fixed node through a non-linear spring tuned to represent the soil behaviour. This technique allowed to simulate the compressive response of the soil, previously calibrated, and to avoid any tensile resistance allowing the footing to uplift. Additional damping related to SSI were introduced as well.

Simulation results of the studied scenarios when SSI is implemented on stiff soil sites, such as Class C site, show that the rocking response can actually serve as an additional energy dissipation mechanism. It can limit the input of the seismic energy into the SFRS leading to an inferior overturning moment imposed on the foundation compared to the SFRS capacity moment. Measured settlements, uplift and rotations do not exceed the recommended values for an adequate response. Lateral deflections and soil bearing stresses remained within the acceptable range to ensure an adequate performance.

The results obtained for softer soils (Class E site), suggest a possible problematic response of the soil-foundation-frame system. Current code provisions were judged necessary for such cases to ensure an acceptable response. If A23.3 rotation related provision or capacity design provisions are ignored, unacceptable settlements were observed.

Smaller foundation dimension can be achieved when site conditions are favourable, reducing construction costs and making steel braced frames an economical and safe bracing option. On the other hand, caution must be taken when site conditions are less favorable.

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## LIST OF SYMBOLS AND ABBREVIATIONS

|                      |                                                                                                                           |
|----------------------|---------------------------------------------------------------------------------------------------------------------------|
| A                    | Area                                                                                                                      |
| $a_0$                | Dimensionless factor for radial damping estimation                                                                        |
| $a_0, a_1$           | Rayleigh damping coefficients                                                                                             |
| $a_1, a_2, a_3, a_4$ | Steel02 material isotropic hardening parameter                                                                            |
| ACI                  | American Concrete Institution                                                                                             |
| $A_g$                | Gross cross-section area                                                                                                  |
| ASCE                 | American Society of Civil Engineers                                                                                       |
| B                    | Maximum of all values of $B_x$ , in both orthogonal directions,                                                           |
| b                    | Width of the footing (parallels to the axis of rotation).                                                                 |
| $b_0$                | Steel02 uniaxial material kinematic hardening parameter (ratio between post-yield tangent and initial elastic tangent)    |
| $b_f$                | Width of the footing (parallels to the axis of rotation).                                                                 |
| BNWF                 | Beam on Nonlinear Winkler Foundation                                                                                      |
| $B_x$                | Ration at level x used to determine torsional sensitivity of a building.                                                  |
| $C'_{prob}$          | Probable post-buckling compressive resistance of a bracing member                                                         |
| CBF                  | Concentric braced frame                                                                                                   |
| CM                   | Center of mass                                                                                                            |
| CP                   | “Capacity protected” foundation type                                                                                      |
| $C_{prob}$           | Probable compressive resistance of a bracing member at first buckling                                                     |
| CQC                  | Complete-quadratic-combination method for modal combination                                                               |
| $C_r$                | Factored compressive resistance o a member                                                                                |
| $C_r, c, n$          | QzSimple1 uniaxial material constitutive parameters controlling the shape of the post-yield portion of the backbone curve |
| $C_r, c, n$          | QzSimple2 uniaxial material constitutive law parameters                                                                   |

|           |                                                                                                                                                  |
|-----------|--------------------------------------------------------------------------------------------------------------------------------------------------|
| cR1, cR2  | Steel02 uniaxial material constitutive law parameters to control the transition from elastic to plastic regions                                  |
| CSA A23.3 | National Standard of Canada: Design of Concrete Structures                                                                                       |
| CSA S16   | National Standard of Canada: Design of Steel Structures                                                                                          |
| $C_u$     | Probable compressive resistance of a bracing member at first buckling                                                                            |
| $C_u'$    | Probable post-buckling compressive resistance of a bracing member                                                                                |
| D/C       | Demand / capacity ratio                                                                                                                          |
| DAA       | Direct analyses Approach for soil-structure interaction implementation                                                                           |
| DBE       | Displacement-based element                                                                                                                       |
| DCE       | Deformation controlled elements                                                                                                                  |
| $D_{nx}$  | Plan dimension of the building at level x perpendicular to the direction of the seismic loading being considered.                                |
| DOF       | Degree of freedom                                                                                                                                |
| $d_w$     | Footing thickness                                                                                                                                |
| E         | Elastic modulus of steel assumed as 200 000 MPa; Earthquake load                                                                                 |
| $E_c$     | Elastic modulus of concrete                                                                                                                      |
| EW        | East-west direction                                                                                                                              |
| $e_x$     | Distance measured particular to the direction of earthquake loading between centre of mass and centre of rigidity at the level being considered. |
| F(T)      | Site coefficient at T period                                                                                                                     |
| FBE       | Force-based element                                                                                                                              |
| FCE       | Force controlled elements                                                                                                                        |
| FEMA      | Federal Emergency Management Agency                                                                                                              |
| $F_t$     | Portion of V to be concentrated at the top of the structure to account for higher vibration modes                                                |

|          |                                                                                                                                      |
|----------|--------------------------------------------------------------------------------------------------------------------------------------|
| $F_x$    | Lateral force applied to level $x$                                                                                                   |
| $F_y$    | Specified minimum stress yielding stress                                                                                             |
| $G$      | Shear modulus.                                                                                                                       |
| $G_0$    | Initial shear modulus of soil or rock                                                                                                |
| GM       | Ground motions record                                                                                                                |
| GMP      | Giuffrè-Menegotto-Pinto uniaxial material                                                                                            |
| GUI      | Graphical User Interface                                                                                                             |
| $H$      | Height                                                                                                                               |
| $h$      | Length of the footing (perpendicular to the axis of rotation).                                                                       |
| $h'$     | Effective structure height measured from the base of the foundation to the center of mass of the first mode. Assumed as 75% of $h_n$ |
| $h_n$    | Height of the structure about ground                                                                                                 |
| HSS      | Hollows Steel Section                                                                                                                |
| $I_E$    | Earthquake importance factor of the building                                                                                         |
| $I_w$    | Wind importance factor of the building                                                                                               |
| $I_y$    | Moment of inertia about $y$ axis.                                                                                                    |
| $J$      | Torsional constant of a section                                                                                                      |
| $K$      | Stiffness                                                                                                                            |
| $k$      | Vertical stiffness of the soil                                                                                                       |
| KL       | Effective length for flexural buckling.                                                                                              |
| $k_x$    | Lateral stiffness of the soil                                                                                                        |
| $k_{yy}$ | Rotation stiffness of the soil                                                                                                       |
| $k_{yy}$ | Rotational stiffness of the footing about the $y$ axis                                                                               |
| $L$      | Length of a given element; live load                                                                                                 |

|                 |                                                                                   |
|-----------------|-----------------------------------------------------------------------------------|
| $\ell_f$        | Length of the footing (perpendicular to the axis of rotation).                    |
| $L_1, L_2, L_3$ | Thornton imaginary columns length                                                 |
| $L_{ave}$       | Average of the length $L_1, L_2$ and $L_3$ shown in figure Figure 4.14            |
| $L_{end}$       | Length of the stiffer end region a footing needed to reflect rotational stiffness |
| $L_f$           | Overall length of the footing                                                     |
| LTB             | Lateral torsional buckling                                                        |
| $L_W$           | Length of the slot in the HSS for gusset-plate insertion                          |
| $M$             | Bending moment                                                                    |
| $M_d$           | Design overturning moment                                                         |
| $M_f$           | Overturning moment that the CBF develops at each analysis step                    |
| $M_n$           | Nominal resistant moment of the SFRS                                              |
| $M_{n,75\%}$    | 75% of the Nominal resistant moment of the SFRS                                   |
| $M_{prob}$      | Probable flexural resistance of a member                                          |
| MR              | Magnitude-distance scenario                                                       |
| $M_r$           | Factored moment resistance of a member                                            |
| $M_{RdRo=2}$    | Overturning moment of the SFRS due with reduced seismic effect by the factor      |
| MRI             | Mean Return Interval                                                              |
| $M_{rot}$       | Overturning moment that trigger the rotation of the footing                       |
| $M_v$           | Higher mode factor                                                                |
| $M_w$           | Moment magnitude                                                                  |
| $M_x$           | Flexural moment about the X axis                                                  |
| $M_y$           | Yielding bending moment                                                           |
| $N$             | Notional load                                                                     |
| NBC             | National Building Code                                                            |

|                                       |                                                                                                                                                        |
|---------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------|
| NCP                                   | Not-capacity protected foundation                                                                                                                      |
| $n_e$                                 | Number of subelements in the entire diagonal                                                                                                           |
| $n_f$                                 | Number of fibers in the cross-section                                                                                                                  |
| $n_i$                                 | Number of integration point at each subelement                                                                                                         |
| NIST                                  | National Institute of Standards and Technology                                                                                                         |
| NS                                    | North-south direction                                                                                                                                  |
| P                                     | Vertical loading                                                                                                                                       |
| PGA                                   | Peak ground acceleration                                                                                                                               |
| PGV                                   | Peak ground velocity                                                                                                                                   |
| $q_0$                                 | Soil yielding resistance                                                                                                                               |
| $q_{\text{fact}}$                     | Factored bearing resistance of the soil                                                                                                                |
| $q_{\text{max}}, q_u, q_{\text{ult}}$ | Ultimate bearing capacity of the soil                                                                                                                  |
| $q_s$                                 | Magnitude of uniform bearing stress in the soil of rock required to resist the applied loads                                                           |
| r                                     | Radius of gyration                                                                                                                                     |
| R                                     | Response factor of a SFRS according with ASCE7-16                                                                                                      |
| R0                                    | Steel02 uniaxial material constitutive law parameters to control the transition from elastic to plastic regions                                        |
| $R_{A23.3}$                           | Footing rotation estimated by following the A23.3 provisions                                                                                           |
| $R_d$                                 | Ductility related force modification. Factor reflecting the capability of a structure to dissipate energy through reversed cyclic inelastic behaviour. |
| $R_e$                                 | Ratio between the end region of the footing and its overall length.                                                                                    |
| $R_k$                                 | Ratio between the spring's stiffness at the end region of the footing and those at the middle region                                                   |
| $R_o$                                 | Overstrength related force modification                                                                                                                |

|                |                                                                                                                                                                                   |
|----------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| $R_{OPENSEES}$ | Footing rotation estimated from OpenSees analysis results                                                                                                                         |
| $R_{sh}$       | Factor applied to probable yield strength to estimate the probable strength including strain hardening                                                                            |
| $R_y$          | Factor applied to $F_y$ to estimate the probable yield stress                                                                                                                     |
| $S$            | Elastic section modulus, Snow load                                                                                                                                                |
| $s$            | Second                                                                                                                                                                            |
| $S(T)$         | Design spectral acceleration, expressed as a ratio to gravitational acceleration, at period $T$                                                                                   |
| $S_a(T)$       | 5% damped spectral acceleration, expressed as a ratio to gravitational acceleration, at period $T$                                                                                |
| $SC$           | Seismic Category assigned to a building based on its Importance Category and the design spectral acceleration values at periods of 0.2 s and 1.0 s                                |
| $SFRS$         | Seismic force resisting system, that part of the structural system that has been considered in the design to provide the required resistance to the earthquake forces and effects |
| $SLS$          | Service Limit States                                                                                                                                                              |
| $SRSS$         | Square-root-of-sum-of-squares modal combination method.                                                                                                                           |
| $SSA$          | Substructure Approach for soil-structure interaction implementation                                                                                                               |
| $SSI$          | Soil-structure interaction                                                                                                                                                        |
| $S_{SP}$       | Spacing of BNWF springs                                                                                                                                                           |
| $STKO$         | Scientific ToolKit for OpenSees software                                                                                                                                          |
| $T$            | Period; fundamental mode of the structure assuming a fixed base                                                                                                                   |
| $t$            | Thickness                                                                                                                                                                         |
| $\tilde{T}$    | Flexible base period                                                                                                                                                              |
| $T/C$          | Tension-compression type of concentric braced frames                                                                                                                              |

|                   |                                                                                                                                                        |
|-------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------|
| $T_1$             | Fundamental lateral period of the first mode of vibration of the building or structure, in s, in the direction under consideration                     |
| $T_{90\%}$        | Period needed to accumulate the 90% of the participative mass                                                                                          |
| $T_a$             | Fundamental lateral period of vibration of the building or structure, in s, in the direction under consideration                                       |
| $T_{a,empirical}$ | Vibration period of the building calculated with the empirical formulation of NBC                                                                      |
| $T_f$             | Tensile force in a member or component under factored load                                                                                             |
| $T_{max}$         | Upper limit for fundamental lateral period of vibration of the building or structure, in seconds, in the direction under consideration                 |
| $t_{plate}$       | Thickness of a given plate                                                                                                                             |
| $T_{prob}$        | Probable tensile resistance of bracing member                                                                                                          |
| $T_r$             | Factored tensile resistance of a member or component; in composite construction                                                                        |
| $T_u$             | Probable tensile resistance of bracing member                                                                                                          |
| $T_x$             | Floor torque at level x                                                                                                                                |
| $U_1$             | Amplification factor to account for second-order effects due to out-of-straightness of an element.                                                     |
| $U_2$             | Amplification factor to account for second-order effects of vertical loads acting on the laterally displaced storey                                    |
| UHS               | Uniform Hazard Spectrum                                                                                                                                |
| ULS               | Ultimate Limit States design approach                                                                                                                  |
| $V$               | Specified lateral earthquake force at the base of the structure                                                                                        |
| $V_d$             | Specified lateral earthquake force at the base of the structure                                                                                        |
| $V_e$             | Lateral earthquake elastic force at the base of the structure                                                                                          |
| $V_{ed}$          | Adjusted lateral earthquake elastic force at the base of the structure                                                                                 |
| $V_s$             | Specified lateral earthquake force at the base of the structure, the shear wave velocity measured in the soil or rock immediately below the foundation |

|                |                                                                                                                                                                                                                                                  |
|----------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| $V_{s30}$      | Average shear wave velocity, in m/s, in the top 30 m of soil or rock                                                                                                                                                                             |
| $V_w$          | Specified lateral wind force at the base of the structure                                                                                                                                                                                        |
| $W$            | Seismic weight of the structure                                                                                                                                                                                                                  |
| $W_w$          | Whitmore section width of the gusset plate with a 45° projection.                                                                                                                                                                                |
| $Z$            | Plastic section modulus                                                                                                                                                                                                                          |
| $z_0$          | Displacement of $qz$ spring at yielding point                                                                                                                                                                                                    |
| $z_{50}$       | Displacement of $qz$ spring at which 50% of the ultimate load is mobilized                                                                                                                                                                       |
| ZLE            | Zero-length element                                                                                                                                                                                                                              |
| $\alpha_s$     | Length of uniform bearing stress in soil or rock required to resist the applied loads                                                                                                                                                            |
| $\alpha_z$     | Stiffness modification factor for radial damping estimation                                                                                                                                                                                      |
| $\beta$        | Damping                                                                                                                                                                                                                                          |
| $\beta_f$      | Foundation damping                                                                                                                                                                                                                               |
| $\beta_i$      | Assumed structural damping with fixed base                                                                                                                                                                                                       |
| $\beta_o$      | Flexible base system damping                                                                                                                                                                                                                     |
| $\beta_z$      | Vertical radial damping                                                                                                                                                                                                                          |
| $\Delta r$     | Roof displacement due to foundation rotation                                                                                                                                                                                                     |
| $\Delta u$     | Structural maximal displacement having an inelastic response                                                                                                                                                                                     |
| $\Delta w$     | Roof displacement with fixed base                                                                                                                                                                                                                |
| $\Delta y$     | Structural displacement at yielding resistance                                                                                                                                                                                                   |
| $\delta_{ave}$ | Average of the displacements at the extreme point of the structure at level produced by the applied lateral force $F_x$                                                                                                                          |
| $\delta_{max}$ | Maximum storey displacement at the extreme points of the structure at level $x$ in the direction of the earthquake induced by the applied force $F_x$ acting at distances $\pm 0.10D_{nx}$ from the centres of mass (CM) at each floor diaphragm |

|            |                                                                                                     |
|------------|-----------------------------------------------------------------------------------------------------|
| $\gamma_s$ | Density of the soil or rock                                                                         |
| $\theta$   | Footing rotation in radians                                                                         |
| $\eta_z$   | Modification factor of rotational stiffness about y axis due to the embedment effect of the footing |
| $\mu$      | Ductility demand                                                                                    |
| $v_s$      | Average effective shear wave velocity in site soil conditions                                       |
| $\phi$     | Resistance factor                                                                                   |
| $\psi$     | Dimensionless factor for radial damping estimation                                                  |
| $\Omega_0$ | Overstrung factor a a given SFRS according with ASCE7-16                                            |
| $\omega$   | Natural frequency                                                                                   |

**LIST OF APPENDICES**

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## CHAPTER 1 INTRODUCTION

### 1.1 Context

In Canada, it is mandatory to meet code requirements for structural design and construction of a buildings in order to ensure a minimum level of safety against all possible demands that may occur in a life span of a structure. The National Building Code of Canada (NBCC) is the model building code that sets out technical provisions for design and construction of new buildings, notably regarding vertical, wind and seismic loads, analysis requirements, design criteria and construction. Provisions of NBCC are harmonized with Canadian material standards for structural design such as *CSA S19 Design of Steel Structures* and *CSA A23.3 Design of Concrete structures*.

Seismic analysis of buildings structures is most commonly done assuming fixed-base support conditions. However, under earthquake loading, foundation supporting the seismic-force-resisting system (SFRS) move and the resulting displacements and rotations can increase displacement demands in the superstructure. For some type of foundations such as, for example, anchored foundation, foundations supported on piles or foundation of buildings with several underground levels, rotations are restrained and the impact of foundation movements on the SFRS can be ignored. For other, such as, for example, isolated shallow foundation, the impact may be large and must be considered in design.

In 2015 edition of Nation Building Code of Canada provisions for seismic design of foundation underwent significant changes. One important change is that for the foundation unrestrained against rotations, it is required to determine the additional displacement of the SFRS that originate from the foundation rotations and consider them explicitly in the SFRS seismic design. This approach to foundation design has been maintained in the 2020 edition of NBCC.

Two options are possible to design free-to-rotate foundation: capacity-protected (CP) and not capacity-protected- (NCP) option. CP foundation must be able to resist the factored vertical loading and the lateral overturning capacity of SFRS. Seismic energy is dissipated primary in the SFRS, which is preferred as the foundation damage and difficult post-earthquake repairs are avoided. In addition, a SFRS with CP foundation should naturally have a more robust response to any unforeseen severe seismic demand, and additional deformation imposed to the superstructure due to foundation rotation are smaller. NCP foundation, previously known in the NBCC as “rocking”

foundations, need not to develop the full capacity of SFRS. Such foundations can uplift and rock, thereby limiting the forces imposed to the superstructure. The superstructure remains the principal source of energy dissipation and must be designed to develop the level of ductility anticipated for SFRS employed.

Above-mentioned requirements for seismic design of foundations have been proposed based on the studies conducted on reinforced concrete shear wall buildings (Adebar et al. 2014). The inelastic mechanism allowed for this SFRS in a single flexural plastic hinge that develops at the base of the structure. Considering that the maximum force demand happens at the same location, it is easier to control the overstrength for such system, and consequently the forces that are used for foundation design. For concentrically braced steel frames, the situation is quite different. Seismic energy is dissipated in braces through tensile yielding and flexural yielding that occur after the braces buckle in compression. The design aims to distribute inelastic demand to all diagonals. When foundation design forces are calculated for such system, the effect of overstrength cumulates and may cause unrealistically large design overturning moments on foundations. Practicing engineers have noticed significant increase in footing sizes which in certain cases become so large as to push construction project stakeholders to choose a concrete seismic-force resisting system rather than a steel one. The same observations were made by Murugananthan and Koboevic (2018, 2019) in the research project investigating the seismic behaviour of 3-storey steel buildings located on C and E class sites in Vancouver in Montreal and designed according to NBCC 2015 and S16-14 and A23.3-14 standards. In these studies, soil-structure interaction (SSI) was represented using a Beam on Nonlinear Winkler Foundation substructure method, and the seismic response of the soil-foundation-superstructure system was examined using nonlinear time-history analysis conducted in OpenSees (1999). Considering that seismic loads substantially increased in the 2020 edition of NBCC and that only one building height that was addressed in this study, there is a need to extend this investigation in order to reach more general conclusions regarding the applicability of current Canadian seismic design procedures to foundations of steel frames.

## 1.2 Objectives

The main objective of this research project is to investigate the impact of soil-structure interaction on seismic design of shallow foundation of steel concentrically braced frames and thereby validate the Canadian design procedure. Only the foundations free-to-rotate will be studied. The attention will be focused on the impact of foundation rocking behaviour to overall structural response and the validation of the current design methods to estimate the rotation of shallow foundation and increased lateral deflection of the frames. The specific objectives can be detailed as follows.

1. Develop OpenSees numerical models using STKO platform of a series of representative buildings for Canadian scenarios that include soil-structure interaction and thereby permit to represent a realistic rocking behaviour when a time-history analysis using ground motions records compatible with NBC design spectrum are performed.
2. Compare the results obtained for buildings designed following NBCC 2020 and CSA S16-19 with those obtained for the buildings designed using NBCC 2015 and CSA S16-14 and discuss differences.
3. Study the response of the structure considering wide range of representative scenarios for typical Canadian steel buildings.
4. Characterize the effect of SSI on the seismic response of the superstructure.

## 1.3 Methodology

To achieve the described objectives a wide range of scenarios for typical CBF steel buildings has been designed in accordance with the current Canadian design provisions. Archetype building were defined in collaboration with practicing engineers from Bush, Bolhman & partners LLP who were the industrial partners on this project. Studied scenarios were selected considering design location, site class, building height, foundation type and code editions. Vancouver and Montreal locations were selected to account for the differences in seismic mechanisms and seismic hazard in East and West Canada. Sites Class C and E were chosen to evaluate the effect that the strength and rigidity of the soil have on the response of the structure. Three- and six-storey buildings were selected for to increase the applicability of the conclusions. In view of the results of previous studies (Murugananthan, 2018), which showed that the impact of foundations rotations on overall inter-storey drifts of studied concentrically braced frames was largely overestimated, the design of

foundations respected all but this design criterion when it imposed an increase of foundation sizes. CP and NCP foundation were evaluated for all scenarios.

Table 1.1. Studied scenarios

| Scenario | Code edition | Number of storeys | Location  | Site class | Foundation type |
|----------|--------------|-------------------|-----------|------------|-----------------|
| 1        | NBC 2020     | 6                 | Montreal  | C          | Fixed           |
| 2        |              |                   |           |            | CP              |
| 3        |              |                   |           |            | NCP             |
| 4        |              |                   |           | E          | Fixed           |
| 5        |              |                   |           |            | CP              |
| 6        |              |                   |           |            | NCP             |
| 7        |              | 3                 |           | C          | Fixed           |
| 8        |              |                   |           |            | CP              |
| 9        |              |                   |           |            | NCP             |
| 10       |              |                   |           | E          | Fixed           |
| 11       |              |                   |           |            | CP              |
| 12       |              |                   |           |            | NCP             |
| 13       |              | 6                 | Vancouver | C          | Fixed           |
| 14       |              |                   |           |            | CP              |
| 15       |              |                   |           |            | NCP             |
| 16       |              |                   |           | E          | Fixed           |
| 17       |              |                   |           |            | CP              |
| 18       |              |                   |           |            | NCP             |
| 19       |              | 3                 |           | C          | Fixed           |
| 20       |              |                   |           |            | CP              |
| 21       |              |                   |           |            | NCP             |
| 22       |              |                   |           | E          | Fixed           |
| 23       |              |                   |           |            | CP              |
| 24       |              |                   |           |            | NCP             |

Once all design tasks were completed, detailed OpenSees models of the designed SFRS were made for each scenario. To economize computational resources, only a 2D frame model was considered. P-delta effects were accounted for by adding an equivalent column to induce stability effects that the vertical load of the whole building may have on the SFRS. Inelastic behavior of steel braces

was represented using fiber cross-section. Yielding and post-yielding strength for braces were introduced by using Giuffré-Menegotto-Pinto hysteretic material Steel02. The effect of rigid nodes and flexibility of connections were explicitly incorporated in the model. Because the elements others than the braces are not expected to have important inelastic displacements, columns, beams and foundation were declared as elastic element congruently with capacity design principles. Next, the soil-structure interaction was modeled using a Beam on Nonlinear Winkler Foundation (BNWF) to reflect the rocking of the overall SFRS and nonlinear soil behaviour. Soil properties was represented using zero-length elements with uniaxial material QzSimple2 from OpenSees uniaxial material library. Then, suites of ground motions records were selected and scaled for nonlinear time-history analyses. The analyses were done using OpenSees program and *Scientific toolkit for OpenSees software* (STKO) to take advantage of its graphical user interface for pre-processing, analysis and post-processing of OpenSees models.

Finally, OpenSees results were used to validated seismic design procedure and appreciate the differences between NBCC 2015 and NBCC 2020 designs. The following response parameters are chosen to describe and compare structural behavior:

- Inter-storey drifts
- Braces force demands
- Building base shear
- Overturning moment developed by the SFRS
- Foundation rotation
- Soil bearing demands
- Soil settlements
- Modal periods shifting

Conclusions and recommendation were formulated based on the obtained results.

The research work was regrouped in the following phases:

Conduct literature review about soil-structure interaction, steel seismic design and non-linear numerical modeling.

Design SFRS for the proposed scenarios following the current Canadian design requirements provided in the CSA S16-19 and the NBCC 2020.

For each scenario design foundations following the procedure for “Capacity-protected” and “Not-capacity-protected” foundation types defined in the CSA A23.3-19.

Validate the modeling assumption by performing analysis on a benchmark model to compare results with previous studies.

Create the STKO numerical model for each studied scenario with and without considering soil-structure interaction.

Select and scale sets of ground motion record representative of the Est and West Canadian seismic hazard.

Perform the nonlinear time-history analyses for all selected records and scenarios.

Compare and discuss the results.

## **1.4 Organisation of the thesis**

This document is organized in six chapters and two Annexes. The first chapter introduces the problem and present the objectives and the methodology undertaken to achieve the objectives. Through literature review, presented in the second chapter, the reader be introduced into the terminology and current knowledge of the studied subjects. The third chapter outlines the design methodology and presents the design of the prototype building for a selected scenario. The fourth chapter describes in detail the methodology employed for the numerical modeling and describes the selection and scaling of ground motion records used in the analysis. The results of the time-history analyses and discussions are presented in the chapter five. Finally, the chapter six summarizes the undertaken work and provides the conclusions and suggestions for further work.

## **CHAPTER 2      LITERATURE REVIEW**

A literature review has been conducted to get familiar with the current state of knowledge on the subject relevant to this project, and to position the findings and conclusion on this research.

The chapter begins with relevant notions regarding earthquake resistant design, addressing fundamental concepts such as ductility, capacity design, yielding mechanism among others. NBCC analysis methods as well as CSA S19 and CSA A23.3 design provisions are also presented. Notions relevant to soil-structure interaction are introduced next, including a discussion on soil mechanical and dynamic properties. Beam on non-linear Winkler foundation concept is then explained.

### **2.1 Earthquake resistant design**

In Canada, all buildings structures must be designed for seismic loads. NBCC puts forward three objectives for seismic design of buildings.

- Preserve the life of the public during high intensity tremors.
- Reduce and limit building damage during weak intensity tremors.
- Guarantee the immediate occupancy and usability of buildings with a post-disaster importance.

While for vertical and wind loads structures are expected to respond elastically, for earthquake loads inelastic response of selected structural elements is desired. These elements are detailed to develop deformation in plastic range while maintaining the resistance and the stability. Other structural elements should exhibit elastic response. This is achieved by applying a capacity design methodology.

A full design process can be summarized as follow. The elastic seismic demands are calculated first, based on the dynamic properties of the structure, such as rigidity, weight and stiffness, as well as geotechnical properties of the site and seismic hazard values at design location. While it is possible to design a building to respond elastically to earthquake loads, the resulting design forces are high and imply an uneconomical project with impractical implications in terms of architectural needs and construction logistics. For these reasons, design forces are reduced to take into consideration an inelastic structural response to earthquake loads. As shown in Figure 2.1 if this

approach is followed, inelastic response of the structure occurs after the lateral strength is reached. Because of that, adequate ductile related provisions are imposed by the steel design standards to ensure an adequate behavior of the building.

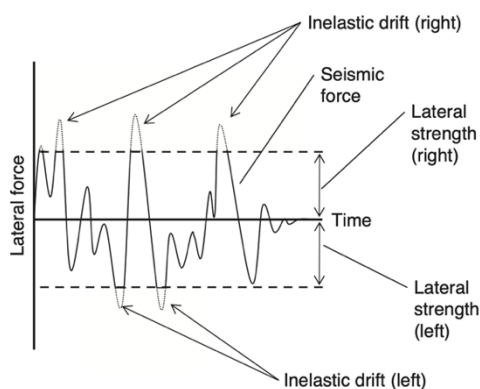


Figure 2.1 Inelastic response is characterized by a limited force developed by the SFRS and by an additional inelastic drift produced by the lack of lateral strength. Adapted from Bruneau et al. (2011).

The NBC introduces  $R_d$ , ductility reduction factor to characterise the ductile response of the structure based on selected yielding mechanisms that may occur during an earthquake, and the overstrength factor  $R_o$  to account for the excess of strength presented in every structure due to differences between analytical properties used by the designer and the actual physical properties of the building, and other causes related to design procedures. Expected elastic forces are divided by  $R_d R_o$  to get design forces (inelastic forces).  $R_d$  and  $R_o$  factors are given in the NBC for different seismic force resisting systems. Restriction in height may apply under certain conditions.

Once the design forces are established and the seismic analysis is performed, *capacity design* approach is implemented, according to material standards (CSA S16 for steel structures). The calculated demands are used only for the design of the “deformation-controlled elements” (DCE) which are strategic elements that develop inelastic behavior through yielding mechanisms, while protecting the rest of the structure which remains elastic during an earthquake. DCEs can be considered as fuses; once their ultimate capacity is reached, no additional force can be transmitted

towards they adjacent elements. They are designed to provide ductility to the structure. For CBFs diagonals are detailed to achieve this purpose. The rest of the elements of the SFRS, called force-controlled elements (FCE) or protected elements (beams and columns) are designed to withstand the expected maximum capacity that the DCE elements can develop.

### **2.1.1 Changes in the 2020 editions of the NBC**

The 2020 edition of the NBC was released recently and was incorporated into this project. This section presents some of most significant changes identified that have a direct impact on the methodology, calculation and results.

#### *Site characterisation and Seismic hazard values*

The 2015 edition of NBC employs the *Canada's 5<sup>th</sup> Generation seismic hazard model* which provides a uniform seismic hazard for all the Canadian territory with a probability of exceedance of 2% in 50 years (0.0000404 per annum). This hazard is presented as spectral acceleration  $S_a(T)$  determined with 5% of critical damping for different fundamental periods considering Site Class C, a reference ground condition. Peak ground acceleration (PGA) and peak ground velocity (PGV) are also available.

To take into consideration the actual conditions of the soil on a selected site, and their impact on the design spectral response acceleration  $S(T)$ , site coefficients  $F(T)$  are introduced. An amplification due to soft site conditions such as clays, silks, soft sands, or a de-amplification due to still site condition such as very stiff soil or rock can be estimated.

For the NBC 2020 edition, the *Canada's 6<sup>th</sup> Generation seismic hazard model* is used which introduces some important changes related to seismic sources for the western Canada, an update in the Ground Motion Models (GMMs) with substantial changes for eastern Canada, and the possibility to make direct calculations for different Site Classes (Adams et al. 2019). Site coefficient are no longer explicitly defined.  $S(T)$  is specified for either a site class A, B, C, D and E, or for a specific site conditions defined by a specific shear wave velocity  $V_{s30}$ . This methodology allows having a continuum hazard for various stiffness characteristics. Specific shear wave velocity,  $V_{s30}$ , estimates being on conservative site, project stakeholders are motivated to make site exploration in order to have more accurate soil characteristics. Seismic hazard tool is available to the public at NRC website.

In the scope of this study a briefly comparison has been made to quantify the impact of these changes on the values of the spectral acceleration for both design locations and sites investigated.

Comparing  $S(T)$  of both codes, it was found that the spectral accelerations are up to 73% higher for Montreal for a specific period, and up to 40% for Vancouver. Despite of these larger differences in  $S(T)$ , the weight of the structures designed using the two editions of the Code did not differ in the same proportions: a difference in the weight around 25% was recorded for both location for site Class E and about 15% for site Class C. Figure 2.2 present the change in  $S_a(0.2)$  between the 5<sup>th</sup> and the 6<sup>th</sup> *Generation seismic hazard model* across de country. Note that eastern regions present a higher rate of change compared with the pacific coast. This implies a greater influence in the design of buildings in Montreal considering the resent edition of NBC.

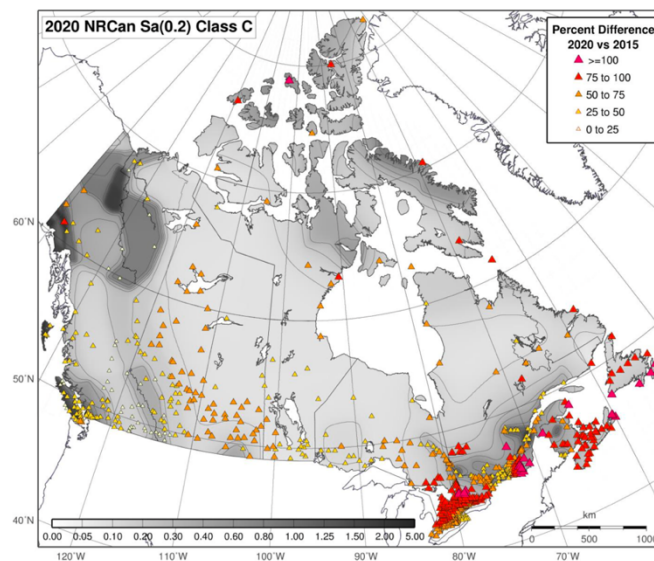


Figure 2.2 Change in  $S_a(0.2)$  hazard from NBC2015. Red-scale triangles indicate the percentage change from NBC2015 for specific localities, while grey shading indicates the 6th Generation  $S_a(0.2)$  hazard across Canada. Adapted from Adams et al. (2019)

### *Seismic design category*

A new seismic design category is implemented by the NBC to quickly categorize the seismic effect on the structure. The category is established by the magnitude of spectral acceleration through structural periods of 0.2 seconds and 1.0 seconds as well by the importance of the structure. Some design provisions may apply depending the seismic design category.

Table 2.1 NBC2020 Table 4.1.8.5-B: Seismic Categories for Buildings. Adapted from NBC (2020)

| Seismic Category | $I_{ES}(0.2)$                     | $I_{ES}(1.0)$                   |
|------------------|-----------------------------------|---------------------------------|
| SC1              | $I_{ES}(0.2) < 0.2$               | $I_{ES}(1.0) < 1.0$             |
| SC2              | $0.2 \leq I_{ES}(0.2) < 0.35$     | $0.1 \leq I_{ES}(1.0) < 0.2$    |
| SC3              | $0.35 \leq I_{ES}(0.2) \leq 0.75$ | $0.2 \leq I_{ES}(1.0) \leq 0.3$ |
| SC4              | $I_{ES}(0.2) > 0.75$              | $I_{ES}(1.0) > 0.3$             |

## 2.1.2 Superstructure analysis requirements

### *Loads*

Dead, live and snow loads are considered as companion loads to seismic load with load factors as specified by NBC. Minimum live loads due to building occupancy are given in Table 4.1.5.3 of the NBC. Live load reduction coefficients can be applied. Seismic load is determined in function of the spectral acceleration based on the uniform hazard spectrum computed with a mean return interval (MRI) of 2500 years which is equivalent to 2% of exceedance in 50 years (0.000404 annually).

The effect of the wind must be considered in the design. In this study, wind forces were estimated by static and dynamic methods following NBC 2020 provisions and in all cases were inferior to earthquake induced forces.

### *Stability requirements*

All structures, regardless the seismic design category, shall have sufficient stiffness to avoid stability issues. P- $\Delta$  effects originating from the storey out-of-plumbness influence unfavorably structural behaviour by reducing lateral resistance of the structure, triggering unsymmetrical response under cyclic loading and increasing lateral displacements. As Figure 2.3 presents, these effects lead to a reduction in the lateral rigidity of the structure. Likewise, at local scale of a single member, geometrical imperfections can lead to a decrease of the compressive resistance by inducing flexural stresses due to a longitudinal dealignment of the element central axis and load application points.

CSA S16 acknowledge these effects by requiring to explicitly considering them through structural analysis and/or design procedures. Several methodologies can be followed to achieve this goal.

- 1) P- $\Delta$ : Perform a non-linear static analysis which achieve static equilibrium at deformed structure.
- 2) P- $\Delta$ : Perform a non-linear static or dynamic analysis in which at each step, the deformed geometry is considerate.
- 3) P- $\Delta$ : Use of the  $U_2$  factor to increase lateral loading effects into the structure.

$$U_2 = 1 + \frac{PR_d\Delta}{Vh} \leq 1.4 \quad (2.1)$$

where:

$U_2$  = Amplification factor to account for second-order effects of vertical loads acting on the laterally displaced storey

$P$  = Vertical loading

$R_d$  = Ductility related force modification factor reflecting the capability of a structure to dissipate energy through reversed cyclic inelastic behaviour.

$\Delta$  = lateral displacement due to  $V$

$V$  = Lateral loading

$h$  = Height of the structure

- 4) Geometrical imperfection: Modify the geometry of the modeled elements to induce internal stress due to these imperfections. For global effects, a misalignment of the columns at each storey by  $L/500$  seems reasonable since it accounts for column leaning tolerances.
- 5) Geometrical imperfection: Apply *notional loads* to “push” laterally the structure in order to recreate the overall geometrical imperfection effect (globally into the building).

$$N_x = 0.005 \sum C_{f,x} \quad (2.2)$$

where:

- $C_{f,x}$  = Total vertical loading at building's level  $x$ .
  - $N_x$  = Lateral notional load at building's level  $x$ .
- 6) Geometrical imperfection: Use of the factor  $U_1$  to account for geometrical imperfections at local scale (strength of a single member).

To perform the design of the archetype buildings, methods (2), (5) and (6) have been used. Methods (2) and (4) were employed in the OpenSees models

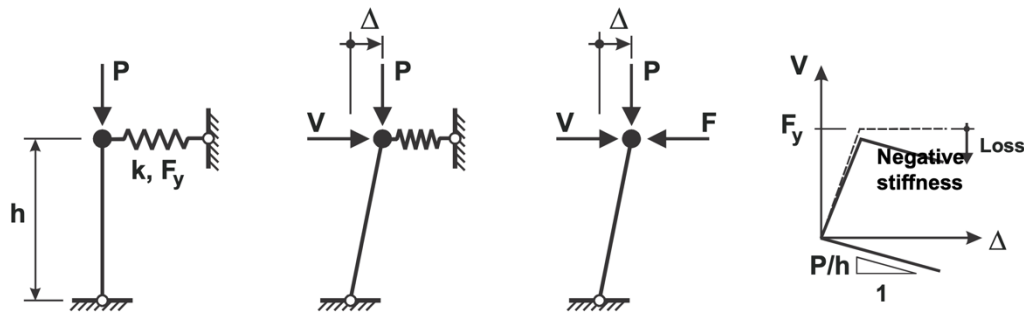


Figure 2.3 Reduction of lateral resistance of the building due to P- $\Delta$  effects. Adapted from Tremblay (2021).

### *Analysis procedures*

NBC proposes two options to perform a seismic analysis: (i) dynamic analysis by either the modal response spectrum method or the numerical integration linear time history method, or (ii) equivalent static force procedure. Both are presented below.

### *Equivalent Static Force Procedure*

In the equivalent static force procedure, the dynamic demands caused by the ground motion are determined by applying a suite of static lateral forces distributed in such way to match the predicted dynamic response of the structure.

Following assumptions are made in this method:

- The first mode of vibration predominates the response of the structure.
- Lateral forces are distributed vertically in accordance with the mass at each level.
- Structural configuration, such as geometrical dimensions and mass distribution are predominantly regular.

These assumptions imply the obligation to test the applicability of the method and limits its use into the following cases:

- Seismic design category is either SC1 or SC2 (low and medium-low intensity)
- Regular structures that are less than 60 m in height and have a fundamental lateral period,  $T_a$ , smaller than 2 seconds in each of the two orthogonal directions.
- Structures with structural irregularities of Type 2, 3, 4, 5, 6 or 8 (see NBC 2020 Table 4.1.8.6) that are less than 2 m in height and have a fundamental lateral period,  $T_a$ , less than 0.5 seconds in each of two orthogonal directions.

For any other case, analysis for earthquake action must be carried out in accordance with the Dynamic Analysis Procedure.

The inputs needed to calculate seismic base shear according to equivalent static force procedure are:

- Seismic weight of the structure,  $W$
- Height of the structure above ground,  $h_n$
- Site classification
- Seismic hazard values
- Type of SFRS
- Importance of the Building ( $I_E$ )
- Vibration period of the first mode of the structure
- Average shear wave velocity in the top 30 m of the soil or rock layers,  $V_{s30}$

The equivalent seismic base shear of the building,  $V$ , is calculated as follows.

$$V = \frac{S(T_a)M_V I_E W}{R_d R_o} \quad (2.3)$$

where:

$S(T_a)$  = 5% damped spectral acceleration

$M_V$  = higher mode factor

$I_E$  = Earthquake importance factor of the building

$W$  = Seismic weight of the structure

$R_d$  = Ductility related force modification

$R_o$  = Overstrength related force modification

NBC 4.1.8.2 defines  $W$  as the dead load of the building considering a minimum of 0.5 kPa for load of partitions and 25% of the snow loading. For storage areas, 60% of live loading shall be included as well as 100% of the weight of the content in tanks.

Because the static methods do not considerate explicitly the complete dynamic response of the structure, higher mode factor,  $M_v$ , is introduced to account for the effect that modes of vibrations different than the first one can have on the base shear. NBC 2020 gives a variety of  $M_v$  values in functions of the building fundamental period, types of SFRS and in the location seismic characteristics expressed by the  $S(0.2)/S(0.5)$  ratio. Intermediate values are obtained by interpolation. As seen in the NBC Table 4.1.8.11, braced frames are not very sensitive to higher mode effects since only structures having a fundamental period near 2 s with  $S(0.2)/S(0.5)$  ratio equal to 70 have a  $M_v$  value greater than 1.

$I_E$  accounts for the importance of the structure and controls the damage presented in the building after a major earthquake.  $I_E = 1$  is used for normal occupancies. Higher value of 1.3 is used for High importance and 1.5 for Post-disaster importance. Table 4.1.2.1 of NBC defines the importance of a given building while Table 4.1.8.5 gives the importance factor for earthquake loading and effects.

$R_d$  is the ductility-related force modification factor that reflects the capability of a structure to dissipate energy through reversed cyclic inelastic behaviour. For moderately ductile CBFs investigated in this study,  $R_d$  is set as 3.0. There are no restrictions of height for seismic design category SC1 and SC2, while the height is limited to 60 m for SC3 and SC4.

$R_o$  is overstrength-related force modification factor accounting for the dependable portion of reserve strength in a structure designed according to NBC 2020 provisions. For moderately ductile CBFs,  $R_o$  takes value of 1.3.

Fundamental lateral period,  $T_a$ , can be determined by either using empirical formulas given in NBC or obtained by modal analysis. For steel braced structures,  $T_{a,empirical}$  can be determined as follows.

$$T_{a,empirical} = 0.05(h_n)^{3/4} \quad (2.4)$$

When a modal analysis is used to determine the fundamental lateral period of the structure, this later shall not be taken greater than  $2T_{a,empirical}$ .

After calculating the fundamental vibration period, the spectral acceleration is obtained from the Uniform Hazard Spectrum (UHS) calculated for a 20%/50 years probability of exceedance and with 5% of critical damping. 2020 Nation Building Code of Canada Seismic Hazard Tool, allows to construct the design spectrum for a given geographical location in Canada considering the soil properties either by Site Class categorization (A to F) or by considering a specific shear wave velocity  $V_{S30}$ . Since this tools only gives the spectral parameters for  $T_a = \{0.2, 0.5, 1.0, 2.0, 0.5, 10.0\}$  intermediate values can be obtained by linear or logarithmic interpolation. Figure 2.4 shows the UHS used in this study.

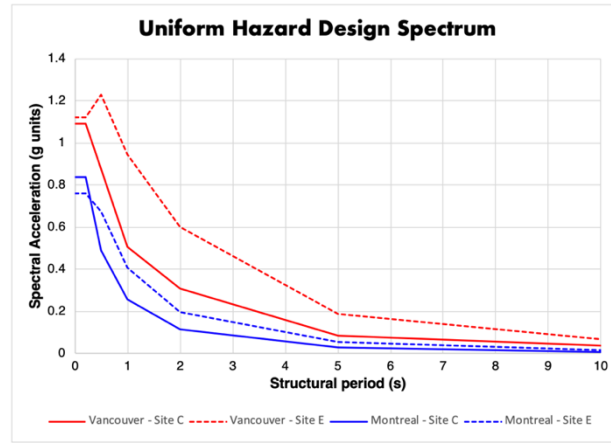


Figure 2.4 Uniform Hazard Design Spectrum from NBC2020 for Vancouver and Montreal, Sites Classes C and E.

NBC 2020 imposes a minimum base shear for steel braced structure in order to ensure a minimum lateral resistance for high periods buildings.

$$V_{\min} = \frac{S(2)M_V I_E W}{R_d R_o} \quad (2.5)$$

An, an upper bound limit is also prescribed, most often applicable for short period buildings

$$V_{\max} = \frac{2 S(0.2)M_V I_E W}{3 R_d R_o} \quad (2.6)$$

$$V_{\max} = \frac{S(0.5)M_V I_E W}{R_d R_o} \quad (2.7)$$

Once  $V$  is calculated, it is distributed over the height of the building in order to have the static equivalent forces at each storey as Figure 2.5 shows. The lateral force applied at storey  $X$  is computed using the following equation.

$$F_x = \frac{(V - F_t)W_x h_x}{\sum_{i=1}^n W_i h_i} \quad (2.8)$$

where  $F_t$  stands for an additional force applied at the top of the building to consider the effect of higher modes of vibration on lateral force distribution.

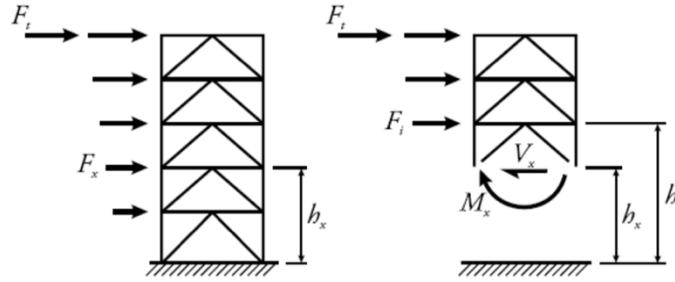


Figure 2.5 Lateral static seismic forces distribution. Adapted from Tremblay (2021).

Furthermore, torsional sensitivity shall be evaluated by calculating the ratio  $B_x$  for each storey  $x$ . The torsional effect of lateral loading being applied eccentrically can be appreciated in Figure 2.6.

$$B_x = \delta_{\max} / \delta_{\text{ave}} \quad (2.9)$$

where

$B$  = maximum of all values of  $B_x$ , in both orthogonal directions,

$\delta_{\max}$  = maximum storey displacement at the extreme points of the structure at level  $x$  in the direction of the earthquake induce by the applied force  $F_x$  acting at distances  $\pm 0.10D_{nx}$  from the centres of mass (CM) at each floor diaphragm.

$\delta_{\text{ave}}$  = average of the displacements at the extreme point of the structure at level produced by the applied lateral force  $F_x$ .

$D_{nx}$  = plan dimension of the building at level  $x$  perpendicular to the direction of the seismic loading being considered.

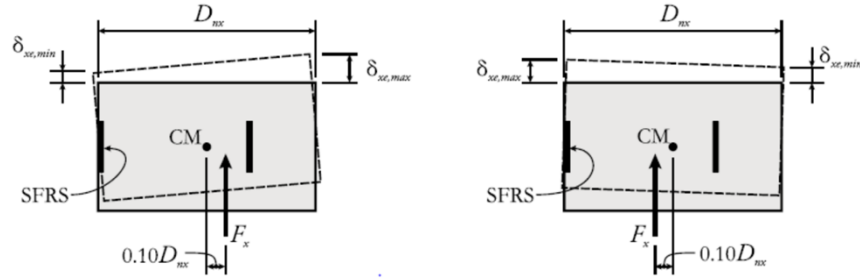


Figure 2.6 Torsional sensitivity. Adapted from Tremblay (2021).

For buildings with  $B \leq 1.7$  or with Seismic Category SC1 or SC2, applied force  $F_x$  shall be placed in each level in such way that it generates a torsional moment  $T_x$  about a vertical axis at each level throughout the building. For other cases, a dynamic analysis procedure is required. The magnitude of  $T_x$  is considering two condition given below and taking the most critical conditions.

- a)  $T_x = F_x(e_x + 0.10D_{nx})$
- b)  $T_x = F_x(e_x - 0.01D_{nx})$

where

$e_x$  = distance measured particular to the direction of earthquake loading between centre of mass and centre of rigidity at the level being considered.

#### *Linear Dynamic procedure: Modal Response Spectrum Method*

The Modal Response Spectrum Method is widely used method to preform seismic analysis of buildings. This method is relatively simple, it offers a high degree of accuracy and it is implemented in numerous commercial software packages.

The method permits to estimate the peak demands of a given structure assuming that the structure will respond elastically in its different modes of vibration. The total responses are then calculated by combining for the responses obtained for each mode of vibration using various combination methods of modal superposition such as *square-root-of-sum-of-squares* (SRSS) or *complete-quadratic-combination* (CQC).

NBC 2020 requires to calculate the structural response by considering the contribution of all modes of vibration whose mass participation sum at least 90% of the total building mass. To account for the inelastic response of the building and its importance, the ordinates of the UHS are divided by the coefficients  $R_d R_o$  and multiplied by  $I_E$  factor and the resulting shear base,  $V_{dyn}$ , is calibrated against the seismic base shear calculated by the Equivalent Static force.  $V_{dyn}$  shall be at least equal to  $V$  calculated using Equivalent Static Method for non-regular buildings and 80% of  $V$  for the case of regular ones. Figure 2.7 presented graphically the meaning of  $R_d$  and  $R_o$  factors.

To account for the torsional sensitivity of the building, an additional static torsional moment about the vertical axis,  $T_x$ , is added to the effects of modal spectral analysis estimated following the same procedure used for Equivalent Static Force Method. For buildings that are not highly sensitive to torsion, i.e.  $B < 1.7$ , it is permitted to use a 3D dynamic analysis with the centre of mass and centre of rigidity shifted by 5% of the plan dimension of the building in the direction of the seismic loading being considered.

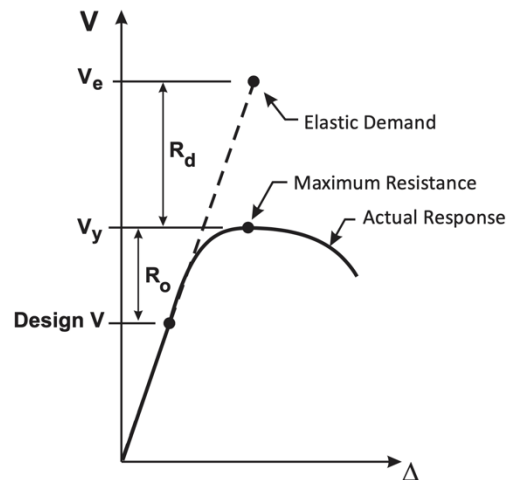


Figure 2.7 Schematic representation of  $R_d$  and  $R_o$  factors. Adapted from Tremblay (2021).

### *Lateral displacements.*

For the elastic analysis methods given in the NBC 2020, lateral inter-storey drifts index shall be limited to 0.010 for post-disaster buildings, 0.020 for high importance Category buildings and 0.025 for all other buildings.

For elastic analysis methods, following the equal displacement rule, presented in Figure 2.8 and in equations 2.10 and 2.11, lateral displacements are estimated from the results of the analysis that includes the torsion effects and multiplied by  $R_d R_o / I_E$  in order to account for the permanent deformation resulting from anticipated inelastic response of the structure.

In addition, starting from 2005 edition of NBC, deflection resulting from the effect of the possible foundation rotation, as shows in Figure 2.9, shall be added to structural drift to get more realistic values of anticipated lateral deflections.

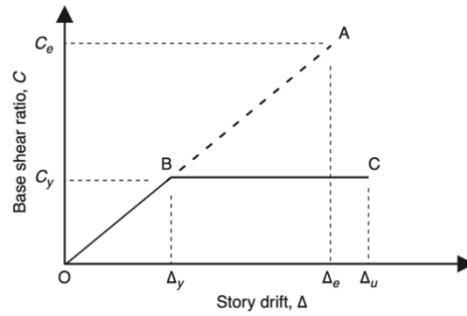


Figure 2.8 Equal Displacements Rule relate the response between elastic and inelastic systems. Adapted from Bruneau et al. (2011)

$$\mu = \frac{\Delta_u}{\Delta_y}; C_y = \frac{C_e}{R_\mu} \rightarrow R_\mu = \mu \quad (2.10)$$

$$R_\mu = \sqrt{2\mu - 1} \quad (2.11)$$

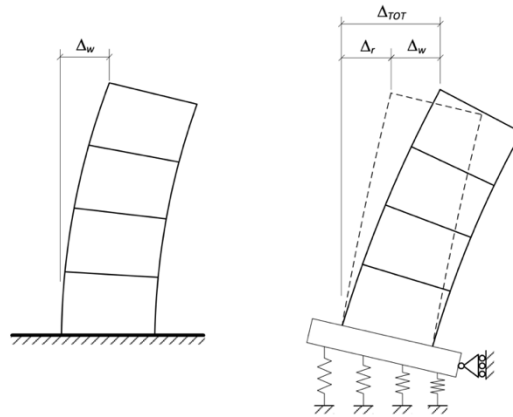


Figure 2.9 Additional lateral deflection due to foundation movements  $\Delta_r$ . Adapted from FEMA P2091 (2020).

### 2.1.3 Superstructure: design provisions

Steel structural design in Canada shall meet all provision specify by CSA S16-19. Capacity design is implemented to achieve a ductile and stable response throughout major seismic tremors. And specific requirements are detailed in Clause 27. As mentioned in previous text, CBF utilises diagonals as DCE elements capable to ensure a stable and ductile hysteretic energy dissipation mechanism while all other components that form part for the SFRS such as columns, beams and diagonals shall remain elastic and must be strong enough to withstand the maximum probable forces that DCE can attain.

The general design provisions S16-19 for earthquake resistant design of moderately ductile CBFs are summarized as follows.

#### *Braces*

Brace profiles are the only elements selected to resist the calculated seismic forces using either of the methods explained in the previous section. Beside seismic solicitations, stability effect such as P- $\Delta$  effect and notional loads, shall also be considered for choosing an adequate cross section while followings CSA S-16 code. Minimum yielding stress ( $F_y$ ) and  $\phi$  factor of 0.9 are used to estimate the resistance of the section under these effects. Vertical effects shall be considered if needed (i.e. chevron configuration). Load combination of NBC 2020 Table 4.1.3.2-A are used to estimate design axial force  $T_f$  and  $C_f$ . Ultimate limit state in yielding shall be checked for tension demands while flexural buckling shall be considered for compression forces.

Because diagonals are DCE elements, additional ductility-related provisions need to be considered. First of all, width-to-thickness ratios limits shall be satisfied in order to avoid local buckling and premature fracture due the formation of plastic hinges at the center of the braces as show in Figure 2.13. Slenderness ratio ( $kL/r$ ) is limited to maximum 200 to ensure a minimum compressive strength. For seismic category SC4, a minimum slenderness ratio of 70 is imposed to allows an adequate out-of-plane flexural buckling which serves as energy dissipation mechanism. This requirement plays an important role for Vancouver design scenarios since demands become high. To meet this requirement, it is suggested to use rectangular HSS profiles instead of square ones in order to increase cross section area of the profile while maintaining a relative low radius of gyration about their weaker axis. Since flexural buckling is highly sensitive to slenderness ratio, the effective length of the diagonal  $kL$  becomes essential. To determinate the effective length, only the portion

of the brace that can physically buckle should be considered. This implies that the offset from column-beam-brace joint to the edge of the elements shall be determined as well as the size of the connection itself. Moreover, it has been shown by Tremblay et al. (2003) that in X-braces if in the intersection of both diagonals is well detailed, the diagonal in tension is able to laterally support the diagonal in compression, reducing the free length of buckling by half as shown in Figure 2.10 and Figure 2.11.

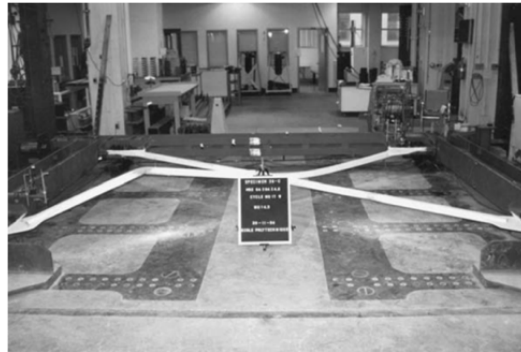


Figure 2.10 Inelastic flexural buckling confined in one segment of X-braces frame during a test. Adapted from Tremblay (2003).



Figure 2.11 Residual inelastic buckling after a strong earthquake of a brace with low slenderness ratio. Note that the tension brace restrains laterally the compressive one. Adapted from Bruneau et al. (2011).

Once the cross-section is selected, probable capacity can be calculated. This capacity represents the maximum force developed by the diagonals when inelastic displacements are imposed. Design is performed using a minimum specified yielding stress ( $F_y$ ) and resistance reduction factor ( $\phi$ ).

However, in the real structure, brace material can have a higher yield stress and can develop a full resistance. In order to account for this, S16-19 requires that a *probable yield stress*,  $R_y F_y$ , be used in calculation of probable resistance in tension ( $T_{\text{prob}}$ ) and compression ( $C_{\text{prob}}$ ,  $C'_{\text{prob}}$ ) as well as  $\phi=1$ . A minimum limit of  $R_y F_y=460$  MPa is imposed for HSS braces and  $R_y F_y=385$  MPa for others profiles.

$$T_{\text{prob}} = A_g R_{\text{sh}} R_y F_y \quad (2.12)$$

$$C_{\text{prob}} = \frac{1.2 C_r}{\phi} \leq A_g R_{\text{sh}} R_y F_y \quad (2.13)$$

$$C'_{\text{prob}} = 0.2 A_g R_{\text{sh}} R_y F_y \leq C_{\text{prob}} \quad (2.14)$$

For CBF  $R_{\text{sh}}$ , a factor to account for strain hardening, is taken equal to 1. In equation 2.13,  $C_r$  is calculated with  $F_y = R_y F_y$ .

Note that in previous editions of S16, a different terminology was used for probable resistances, notably  $T_u$ ,  $C_u$ ,  $C'_u$ . Within the scope of this study, previous terminology is used in the text. Another change introduced in the current edition of S16 is the explicitly description of nominal lateral capacity of SFRS. For such purpose, probable yielding stress,  $R_y F_y$ , is taken equal to the minimum yielding stress,  $F_y$ . Typical hysteretic behaviour of CBF is presented in Figure 2.12. Note that capacity of the elements,  $T_u$ ,  $C_u$ ,  $C'_u$  envelope the response of the elements in terms of force.

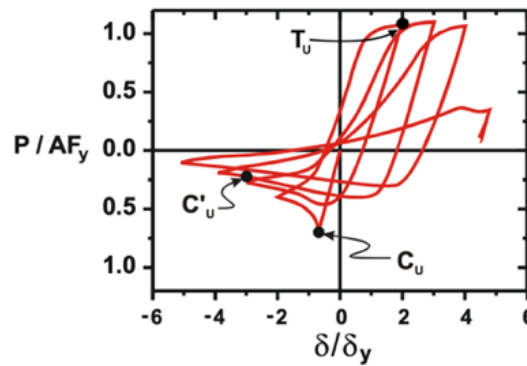


Figure 2.12 Typical hysteretic behaviour of ductile steel braces. Adapted from Tremblay (2021).

In CBFs, diagonals provide two inelastic energy dissipating mechanisms namely yielding in tension and flexural-buckling in compression. As seen in Figure 2.15 shows, the response in compression and tension is not symmetrical. The response during tensile is characterised by higher lateral resistance while during compressive demands more significant displacements are developed. As a result, an adequate overall response of the structure must consider both mechanisms simultaneously. For this reason, at least two diagonals, one in each direction, are mandatory to achieve an adequate response under seismic load with changing direction. Furthermore, as shown in Figure 2.14, diagonals directly connected to the columns are prohibited in order to avoid forces concentration in the columns that could lead to the formation of plastic hinges in these elements, a behavior not anticipate in design.

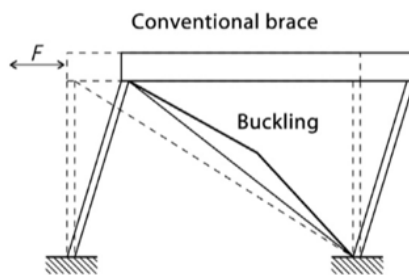


Figure 2.13 Schematic representation of buckling of a concentric the brace under compressive forces. Note the formation of a plastic hinge at the center of the brace. Thick-to-width ration shall be imposed to avoid rupture at the plastic hinge. Adapted from Largo et al. (2019).

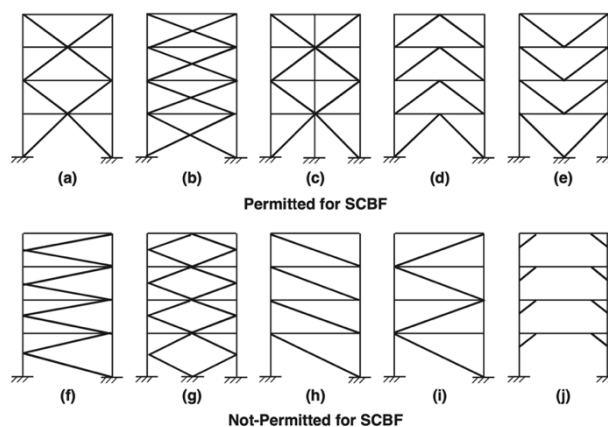


Figure 2.14 Permitted and Not-Permitted diagonals configuration for CBF. Adapted from Bruneau et al. (2011).

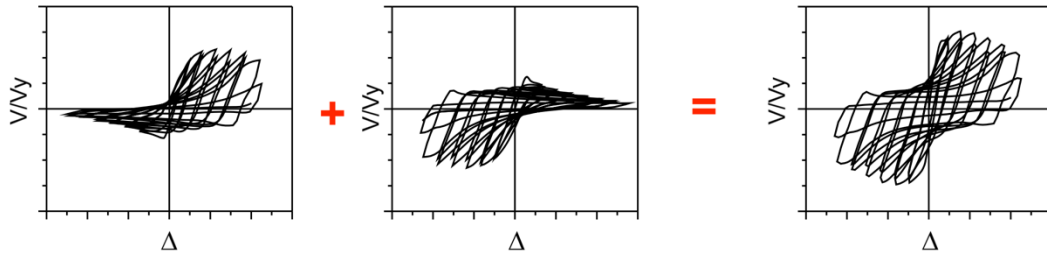


Figure 2.15 Symmetrical response shall be privileged. Adapted from Tremblay (2021).

### Columns and beams

To ensure a favourable hierarchy of yielding mechanisms, other frame elements, such as beams, columns connections and foundations, are designed to sustain probable capacity of diagonals. Corresponding forces are distributed, as shown in Figure 2.16, throughout the rest of the SFRS to determine design forces in these FCE elements. In this way inelastic deformations are confined into the diagonals, protecting the other frame elements which are also called *protected elements*.

In order to avoid any brittle mode of failure, such as local buckling, class 1 or 2 are mandatory for these elements. Additionally, columns must be design to a moment of at least  $0.2ZF_y$  applied in the direction in which earthquake effect is considered.

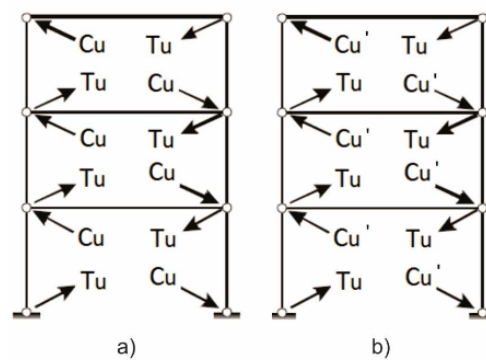


Figure 2.16 Probable capacity developed in the diagonals. Beams and columns shall be designed to withstand probable forces all at once. Adapted from Muruganathan (2018).

### *Connections*

Capacity design must be implemented for the connexion detailing. Connections are designed to be stronger than the members they connect (strength-related criteria) and at the same time be ductile enough to allows inelastic response when out-of-plane brace flexural-buckling is triggered on the diagonals (ductile-related criteria). Diagonals of CBFs can be connected to the rest of the frame in different ways and the final connection configuration is affected by several factors such as architectural concept, architectural size limitation, erection methods, erection logistics, stock availability, labor and machinery availability, etc. For the frame of this study, a welded gusset plate with slotted HSS was selected to reflect the effect of the connection stiffness at both ends of braces in the overall response of the structure.

Shop-welded gusset plates with slotted HSS, similarly as portraited in Figure 2.17 and in Figure 2.20, is one of the most popular connection configurations for CBFs. Usually gusset plates are shop welded to the column, and HSS diagonals are installed on-site by introducing the gusset plate through a previous detailed slot. Site welds are placed once the diagonals are put in place.



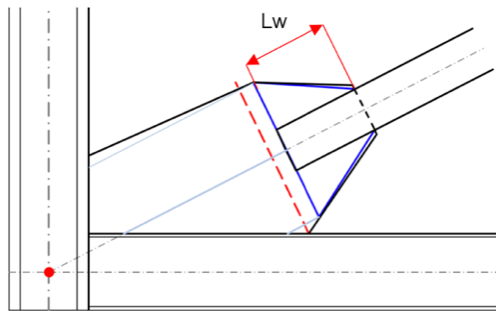
Figure 2.17 Slotted HSS. Reyes (2018).

The importance of the connection design in the frame of this study lies is unfold. On one hand, the size of the connection affects directly the brace compression capacity since larger gusset plates reduce the brace effective length. On the other hand, the flexural out-of-plane stiffness and rotational stiffness of the connection at the ends of braces impact significantly the seismic response of CBFs. Po-Chien Hsiao et al. (2012).

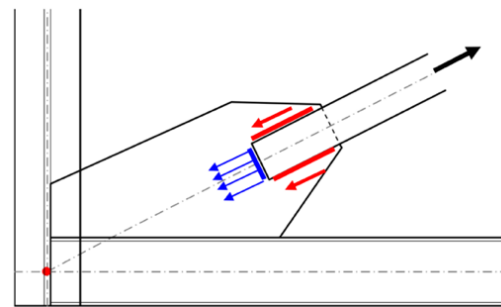
Most important limit states and criteria affecting the plate geometry are summarized below and schematically presented in Figure 2.18 and Figure 2.19.

*Strength-related criteria:*

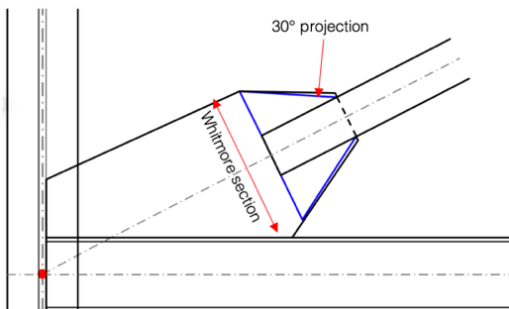
- Slot length  $L_w$  shall be larger enough to allow welds to develop the full tensile capacity of the brace and to avoid tear-out of the HSS wall. As common practice  $L_w$  is set as two to three times the width of the HSS.
- Plate thickness shall be sufficient to resist block shear failure
- Plate thickness shall be sufficient to provide tensile resistance in the Whitmore section.
- Plate thickness shall be sufficient to avoid flexural buckling.
- Plate thickness shall be sufficient to resist the simultaneous action of interface forces at plate-to-column and plate-to-beam sides.



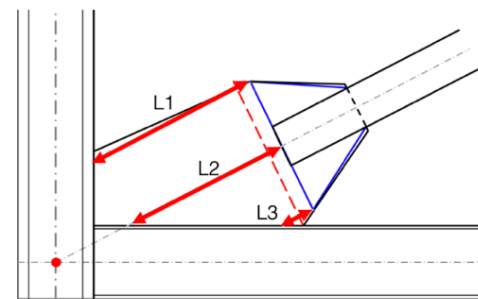
a) Slot length



b) Shear block failure



c) Whitmore section with 30° projection



d) Flexural buckling is checked analogically as column with  $L$  length equal to the average of  $L_1$ ,  $L_2$ ,  $L_3$ .  $K$  factor is assumed as 1.2

Figure 2.18. Gusset plate connections limit states.

### *Ductile-related criteria*

- Free length to allow a formation of plastic hinge. In order to facilitate out-of-plane inelastic displacement once the flexural buckling of the brace is triggered, a free length between the brace and the most rigid part of the connection is recommended so that a plastic hinge is accommodated at the end of the brace. More details on this subject will be provided in the section 2.3.1.
- Slot reinforcement. Because slotted action debilitates the cross section of the braces, a reinforcement plate is required to avoid premature failure before the brace reaches  $T_{prob}$ .
- Free edge stiffeners. They can be needed if the free edge is too long. Note that stiffeners shall not interfere with the free length discussed previously.

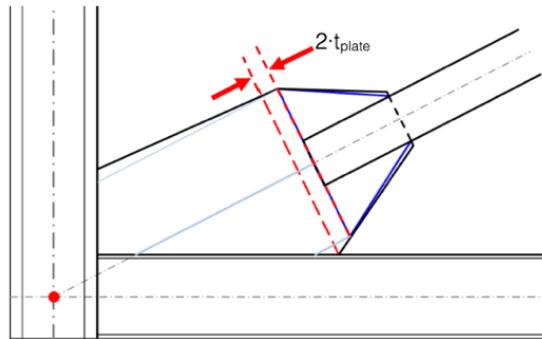


Figure 2.19 Free length of 2 times gusset plate thickness to allow plastic hinge formation.



Figure 2.20 Free length and reinforcement plate. Reyes (2018)

### 2.1.4 Seismic design provisions for foundations

It is a common practice in Canada to use a single shallow footing for both columns of the braced frame for low- and mid-rise CBF buildings. Vertical loading is transferred from the columns to the footing and then spread out into the soil beneath it within a contact area that need to be larger enough to avoid soil failure. On the other hand, for lateral loads, such as earthquake or wind loads, a force-couple is developed in the columns of the CBF causing an overturning moment on the foundations. For calculation purposes, this overturning moment is treated as an eccentric vertical force placed in such position that would have the same effect over the foundation and soil. To illustrate this procedure, Figure 2.21 shows the foundation forces and stresses diagrams need to be determined foundation demands.

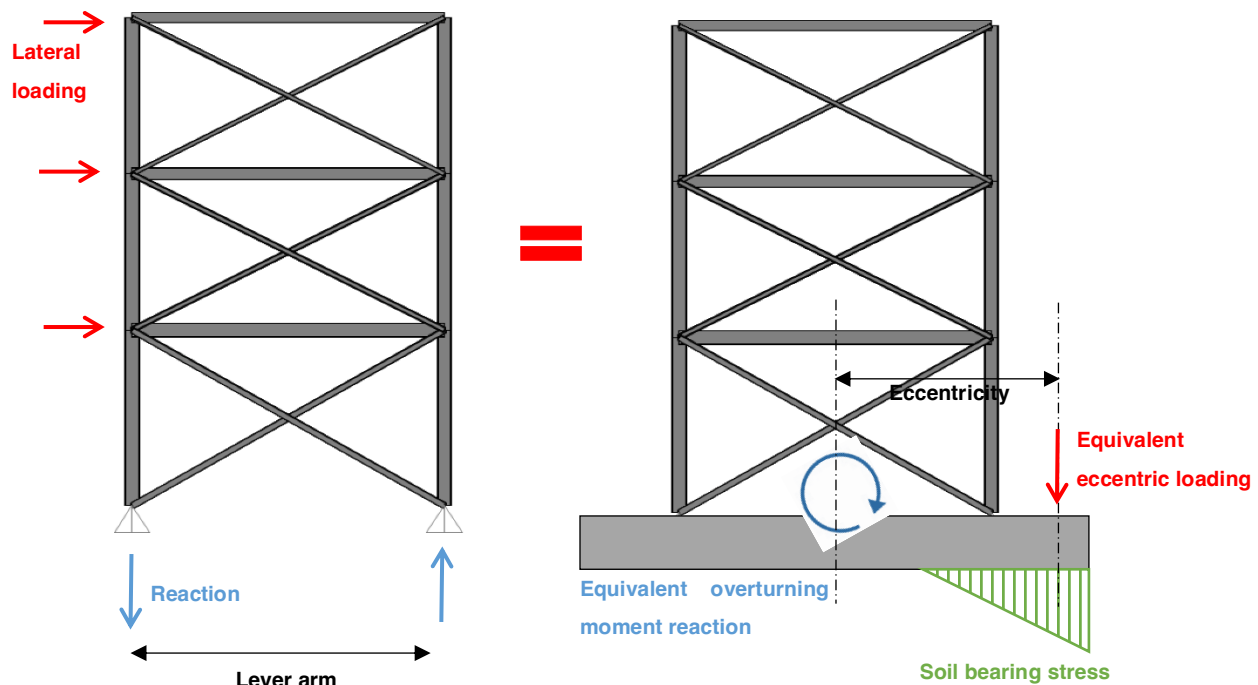


Figure 2.21. Foundation forces and stresses diagrams.

Eccentricity,  $e$ , can be estimated by following expression:

$$e = \frac{M}{P} \quad (2.15)$$

where:

$M$  = overturning moment

$P$  = vertical loading considering the weight of the foundation, overburden due to soil over the footing, and loads of the structure.

As can be seen from this equation, when vertical loading increases, the eccentricity decreases, indicating that vertical loads have a stabilizing effect acting like ballast. That why, for seismic design of steel braced frame footings purposes the NBC load combination of  $1.0D + 1.0E$  shall be used instead of  $1D + 1.0E + 0.5L + 0.25S$ .

In the process of foundation design, four main design criteria have to be verified to ensure a proper behavior under vertical and lateral loadings: (i) overall stability, (ii) soil bearing capacity (iii), soil settlements and (iv) foundation resistance.

Stability is achieved when the eccentric force falls inside of the footprint of the foundation. So, as first step of the design process, an initial dimension of the footing is selected and modified until the stability is assured.

Next, the maximum soil stresses ( $q_{\max}$ ) are checked to guarantee that they do not surpass the factored bearing capacity of the soil  $q_{\text{fact}}$ . As per the Canadian Foundation Engineering Manual (2006) suggests, Limit States Design (LSD) philosophy is used, both for superstructure and foundation calculations. A resistance factor,  $\phi$ , equal to 0.5 is applied to the ultimate bearing capacity ( $q_u$ ) of the soil to estimate its factored bearing resistance.

$$q_{\max} \leq q_{\text{fact}} \quad (2.16)$$

$$q_{\text{fact}} = \phi q_u \quad (2.17)$$

Stress distribution beneath foundation ( $q_{\max}$ ) can be estimated using elastic methods. In the absence of anchors, the soil under the shallow foundation cannot withstand the tensions, therefore this methodology applies only in the case where the eccentricity is located in the mid-third of the length of the foundation,  $h$ , (i.e.  $0 < e < h/6$ ). Figure 2.22 shows the distribution of soil stresses in function of vertical load eccentricity, starting from the condition with vertical load only (2,22 b), up to a full plastification of the soil under the footing, induced by a vertical load and an overturning moment (2.22 f). The equations to estimate  $q_{\max}$  for each case is given in the same figure.

When  $0 < e < h/6$  elastic distribution of stresses is estimated as follows:

$$q_{\max} = \frac{P}{A} \pm \frac{M_x}{S_x} \quad (2.18)$$

where:

$P$  = vertical loading.

$A$  = contact area between the footing and the soil.

$M_x$  = overturning moment about transversal axis of the footing.

$S_x$  = Elastic section modulus about the transversal axis of the footing.

Thus, under sustained loads, such as vertical loads scenarios, the failure of the foundation is most often associated with excessive settlements rather the soil bearing failure. However, under lateral loading such as wind and earthquake, which are transient in nature, capacity of the soil is main design criterion. (Moehle 2015). For transient loading a controlled amount of uplift of the foundation is permitted. Equation has been developed to estimate the stress distribution for larger eccentricities. Trial dimension of the footing are refined to meet both stability and bearing capacity in an iterative process.

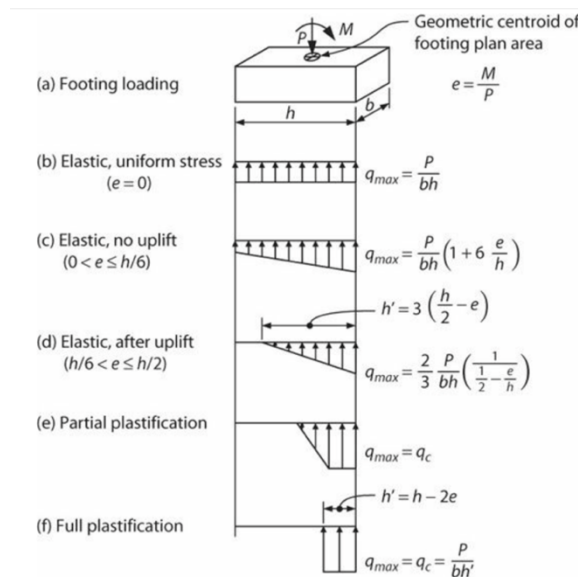


Figure 2.22 Stress distribution beneath shallow foundation. Adapted from Moehle (2015).

Once the stability of the foundation is granted and the bearing stress are within acceptable limits, settlements needs to be verified. Excessive settlements, especially differential ones can induce an adverse effect on the super structure ranging from uneven surfaces, visible cracks, to creating large internal forces that structural elements cannot sustain. Canadian Foundation Engineering Manual recommend the following limits for low risk structures and encourages a constant communication between structural and geotechnical engineers to set the proper settlements limits. Figure 2.23 presents different types of damage according with the nature of the settlements.

Table 2.2 Guidelines for limiting settlements of framed buildings. Adapted from Canadian Geotechnical Society (2006).

| Type of damage                   | Criterion          | Limiting value                      |
|----------------------------------|--------------------|-------------------------------------|
| Structural damage                | Angular distortion | 1/150 – 1/250                       |
| Cracking in walls and partitions | Angular distortion | 1/500<br>1/1000 – 1/15000: end bays |
| Visual appearance                | Tilt               | 1/300                               |
| Connection to services           | Total settlements  | 50-75 mm: sands<br>50-175 mm: clays |

According to ASCE-7 differential settlement limits for multistorey steel buildings, expressed as permitted angular distortion vary between 1/100, 1/166, 1/500 for risk category I or II, III and IV, respectively.

Other recommendations through the literature give similar values for limiting angular distortions. Bjurum (1963) suggest 1/500 for cracking control, 1/250 for visible tilt and 1/150 for avoid structural damage and 25 mm has been used a practical limit for total settlements.

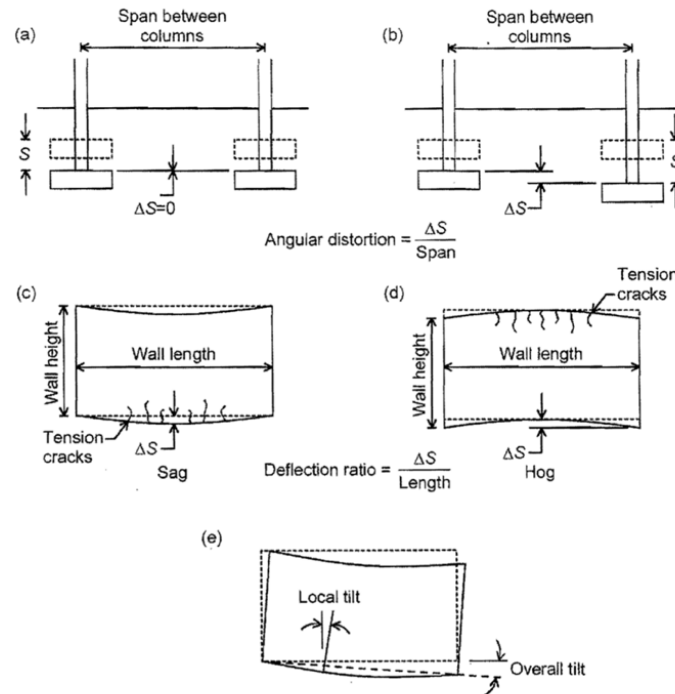


Figure 2.23 Types of tolerable settlements for shallow foundation. Adapted from Canadian Geotechnical Society (2006).

Lastly, the resistance of the footing itself has to be evaluated to resist all the internal forces. Footing depth is mostly governed by unidirectional and bidirectional shear requirements and in some cases additional shear steel reinforcement can be used to avoid extreme footing dimensions. Longitudinal steel bars are provided to assure ductility and flexural strength.

#### *Design provisions*

Since the foundation of steel braced frames are normally made with reinforced concrete, the Special Provisions for Seismic Design of footing given in CSA A23.3 may apply. These provisions make a distinction between foundation restrained to rotate and those that are not. Because only foundation that are free to rotate are considered in this study, literature review addresses only this type of foundation. These types of foundations are schematically presented in Figure 2.24.

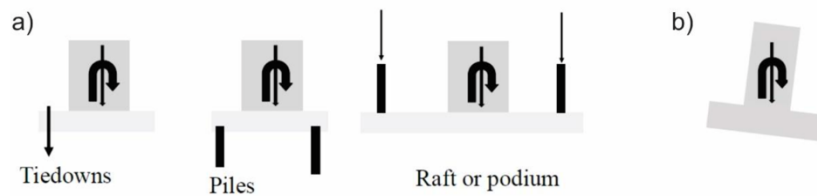


Figure 2.24 Types of foundations. (a) Rotation restrained, (b) Free rotation. Adapted from Murugananthan (2018).

Free-rotating foundations, can be designed using one of the three approaches described in A23.3-19: (i) capacity protected (CP), (ii) Not-capacity protected (NCP) and (iii) Bailout foundation.

### *Overturning moment*

Design of capacity protected foundations follows fully the capacity design philosophy. They are sized to resist the full overturning moment corresponding to the full capacity of a SFRS combined with the action of vertical loads. This objective of such design is to provide a strong foundation so that the energy dissipation under earthquake loads takes place only in the superstructure. Such foundations are expected to have an elastically linear response, while the inelastic demands are induced into the SFRS (Adebar 2014).

As per S16-19, for steel SFRS with  $R_d \leq 2.5$  nominal moment ( $M_n$ ) of SFRS shall be used as design moment ( $M_d$ ) and for systems with  $R_d > 2.5$ , probable moment ( $M_p$ ) should be used in design. This is based in the assumption that more ductile systems are capable to sustain larger inelastic demands. (Metten 2017).

For a moderately ductile CBFs, probable moment is computed by assuming that all diagonals attain their probable capacity at the same time as illustrated in Figure 2.25.

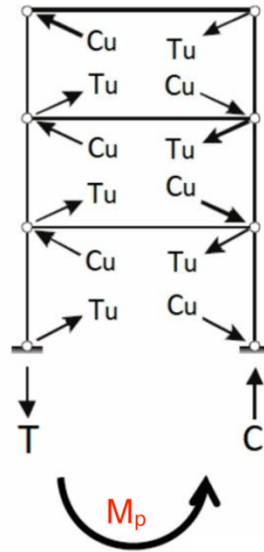


Figure 2.25 Probable moment due to brace's probable forces being developed. Adapted from Murugananthan (2018).

Not-capacity protected foundations do not need to develop the full capacity of the SFRS and thus are sized using a smaller design moment compared to CP foundation. Because the dimension of NCP foundations is not sufficient to develop the probable moment of the SFRS, the rotation causing the settlement of the “toe” of the footing and the uplift of the “heel” is triggered. While the inelastic response of the superstructure remains a principal energy-dissipation mechanism, it was shown that this movement can lead to an additional energy dissipation mechanism for the building during strong ground motions and serves as barrier for the energy input from ground to structure (Lemieux 2014).

Design moment for this type of foundations is taken as the greater value between the 0.75% of the nominal overturning moment capacity ( $M_{n,75\%}$ ) of the SFRS and the overturning moment resulting from design load combinations that include earthquake effects calculated using  $R_d$   $R_o$  equal to 2.0 ( $M_{RdRo=2}$ ).

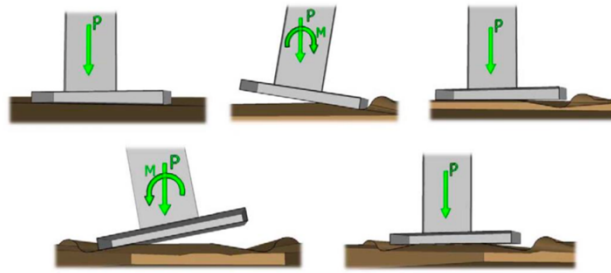


Figure 2.26 Footing rotation cycle. Adapted from Lemieux (2014).

The American foundation design provisions, ACI-318 2019, do not mandate to consider any overstrength or capacity design approach to evaluate the overturning design moment,  $M_d$ . In fact, the moment obtained directly by the analysis including the reduction factor  $R$  (equivalent to the Canadian  $R_d R_o$ ) is permitted to be used for design. Practices in individual offices vary significantly as a function of experience and requirements from the client (Moehle, 2015). Due to the inherent difficulties for foundation inspection and retrofit, engineers tend to apply certain overstrength factor in order to minimize inelastic response of foundation. In some instances, capacity design principles are applied to a certain extent with the objective to control some brittle failure modes related to shear and axial compression. On rare occasion, for buildings with Risk Category III or IV (equivalent to Canadian high-importance or post-disaster importance categories) design offices would opt for a “strong foundation” approach which resembles to Canadian methodology for CP foundation design.

Due to the lack of mandatory procedures for foundations capacity protected design, followings methodologies are suggested to control inelastic response of foundations. (Moehle, 2015).

Increase the overturning moment from analysis by an overstrength factors, typically taken between 1.25 and  $\Omega_0$  of the SFRS.

Use a different reduction factors,  $R$ , for superstructure and for the foundation. Commonly  $R=6$  is used for foundations.

Estimate the capacity developed by the SFRS (as Canadian procedures mandates).

Perform a non-linear time history analyses to evaluate realistic demand on foundations.

Once design forces are determined, foundation design follows usual requirements given in CSA A23.3 The following limits states shall be evaluated by the engineer:

Unidirectional shear (One-way shear)

Bidirectional shear (Two-way shear)

Flexural reinforcement.

#### *Foundation rotation*

As discussed in Chapter 2, foundation movements need to be considered when estimating lateral deflections of the superstructure. As per A23.3 footing rotation recommends using detailed analyses such as inelastic time-history analyses including soil-structure interaction modeling. However, in practice such sophisticated analyses are rarely employed for typical buildings. It is more common to use A23.3's simplified method to estimate footing rotation caused by the "heel" uplift that arises when the eccentricity ratio,  $e/h$ , exceeds  $1/6$  as illustrated in Figure 2.22.

For CP foundations A23.3 gives a following equation to estimate foundation rotation:

$$\theta = 0.3 \left( \frac{q_s}{G_0} \right) \left( \frac{\ell_f}{\alpha_s} \right) \left\{ 1 + 2 \left( \frac{\alpha_s}{b_f} \right)^{1.5} \right\} \quad (2.19)$$

where:

$\theta$  = footing rotation in radians.

$\alpha_s$ : length of uniform bearing stress in soil of rock required to resist the applied loads.

$q_s$  = magnitude of uniform bearing stress in the soil of rock required to resist the applied loads.

$G_0$  = initial shear modulus of soil of rock, which may be estimated in kPa units from  $(\gamma_s V_s^2)/1000$  when,  $\gamma_s$  the density of the soil or rock, is in  $\text{kg/m}^3$ , and  $V_s$ , the shear wave velocity measured in the soil or rock immediately below the foundation, I in units of m/s.

$b_f$  = width of the footing (parallels to the axis of rotation). Previously denoted as  $b$ .

$\ell_f$  = length of the footing (perpendicular to the axis of rotation). Previously denoted as  $h$ .

These parameters are presented graphically in Figure 2.27.

For the structures with NCP foundations, the equation 2.19 can also be used to determine footing rotations but the inter-story drifts of a superstructure, caused by footing rotation must be increased at every level by the greater of the following values:

1. 50% of the displacement at the top of the SFRS determined from a fixed-base model, divided by the height above the footing;
2. The rotation of the foundation calculated using Equation 2.19 when subjected to an overturning moment equal to the nominal overturning capacity of the SFRS; but need not exceed 3.0 times the rotation of the foundation when subjected to an overturning moment equal to the factored overturning resistance of the foundation; or an inter-storey drift ratio equal to 0.005.

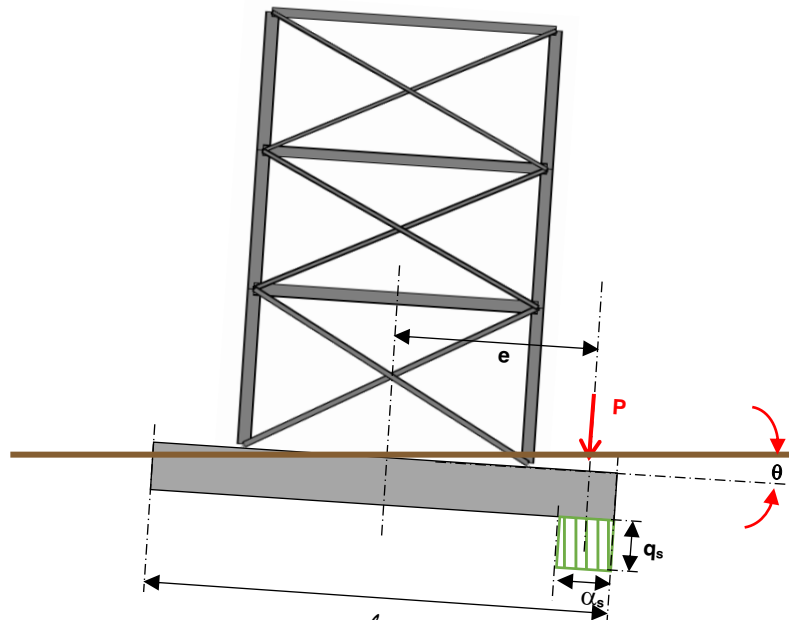


Figure 2.27 Parameter of A23.3 equation 2.19.

## 2.2 Soil-Structure Interaction

Soil-Structure Interaction (SSI) is a process in which, under applied loads, the repose of the soil beneath the structure influence the response of the structure and vice-versa. SSI analyses evaluates the combined response of the structure, foundation and underlying soil. In this analytical approach soil and foundation parameters are explicitly introduced into the mathematical model of a given structure to estimate in a more realistic way its response under static or dynamic demands. Such model is better suited compared to that in which the structure is considered to have a fixed base, leading most often to lower demands estimates. For this reason, it is often used for assessment and retrofit design of existing buildings. Often viewed as a complex procedure for a typical project in engineering offices, it was seldomly used in the past for seismic design of new buildings structures. However, due to recent developments in analytical methods and modelling, availability of general references and commercial software packages, its implementation has been steadily increasingly.

### 2.2.1 Implementation and methods.

SSI consideration can be implemented to any desired building. Nevertheless, some structures may show significantly different response if SSI is considered. NIST 2012 and FEMA 2020 summarize cases for which the inclusion of SSI can play an important role: large building footprint, substantial foundation embedment and high structure-to-soil stiffness ratio. An example of how SSI can influence the response of the structure is illustrated in Figure 2.29.

Considering that the CBFs structures have in general higher stiffness compared to other steel systems, *high structure-to-soil stiffness ratio* is the parameter which can help detect if the deformations of the superstructure will be affected by the soil.

NIST 2012 proposes to classify a given structure as sensitive to SSI effects considering the *high structure-to-soil stiffness ratio* if the following expression is met:

$$\frac{h'}{v_s T} > 0.1 \quad (2.20)$$

where

$h'$  = effective structural height measured from the base of the foundation to the center of the mass of the first mode.

$T$  = fundamental mode of the structure assuming a fixed base condition

$v_s$  = average effective shear wave velocity in site soil conditions.

This expression has been discussed by Bielak (1974), Veletsos and Nair (1975) and Dobry (2014).  $h'/T$  represents the structure stiffness given in the units of velocity to permits a comparison with soil's waves velocity. In general terms, it expresses the structure-to-soil stiffness ratio. By inspection, Figure 2.28 shows that for given  $h'$  and  $v_s$  values, if  $T$  decreases, the ratio become bigger and thus more susceptible the structure may be to SSI effects. For softer soils the ratio increases indicating a more significant impact of SSI on the structural response.

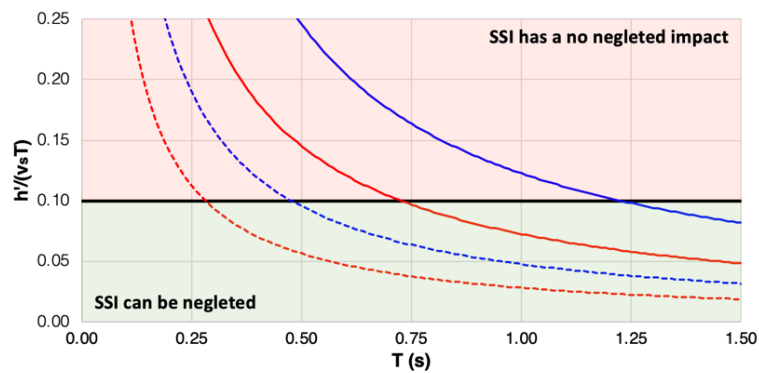


Figure 2.28. Structure-to-soil stiffness ratio for studies scenarios.

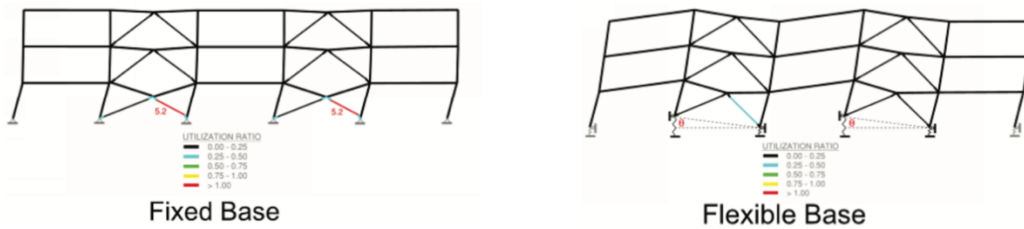


Figure 2.29 The significant impact of soil flexibility on a coupled CBF can change the demand-to-capacity ration of structural elements. Adapted from FEMA (2019).

Stewart et al. (1999) took measurements from monitored buildings that underwent ground shaking and validated, among other, that the structure-to-soil stiffness ratio is the most significant and sensitive parameter to estimate the impact of SSI on structural seismic response. They concluded it can be neglected when the ratio is less than 0.1.

There are two wide-spread methods to implement SSI into analysis: (1) Direct Analysis Approach (DAA) and (2) Substructure Approach (SSA). They are illustrated in Figure 2.30 and Figure 2.31, respectively.

When using DAA a complete system is modeled using finite element method. Soil is represented as a continuum medium using either shell elements for 2D analyses or solid elements for 3D analyses with linear representation of soil properties. A refined mesh is detailed in the surroundings of the foundation while a coarse mesh can describe the soil response in the edge of the model where boundary elements are placed.

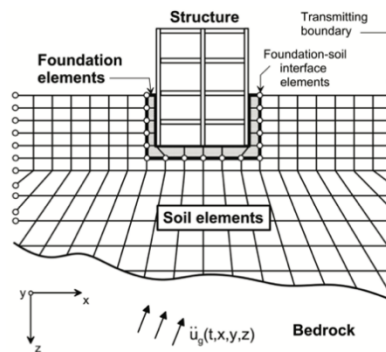


Figure 2.30 Schematic illustration of DAA using continuum modeling by finite elements. Adapted from NIST 2012.

Because such models are computationally demanding and it is rather complex to incorporate kinematic interaction due to spatially variable input motion, this method is rarely used in practice and engineers opt more frequently for SSA. The reader is invited to consult literature for more detailed information on DDA approach.

SSA method is more simple and easier to implement in practice. The soil is modeled using a series of springs beneath a discretized elastic beam which represent the foundation. Superstructure is connected to such beam, and connected to a fully restrained node.

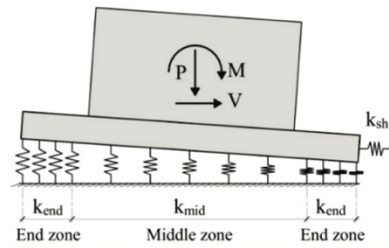


Figure 2.31 Substructure approach using springs that characterize the soil properties. Adapted from FEMA (2020).

### 2.2.2 Beam On Non-linear Winkler Foundation (BNWF)

One of the most common methodologies used for Substructure Approach modeling is known as *Beam On Non-linear Winkler Foundation (BNWF)*. In this methodology, the foundation is modeled, discretized and supported by a series of finite stiffness non-linear springs instead of providing fully rigid support. The properties of these springs are calibrated in function of the soil stiffness and foundation geometry. Construction of the numerical model of the foundations is relatively easy since the springs behave by only one degree of freedom and the foundation itself is often represented as an elastic beam element.

In addition to its simplicity, a major advantage of this methodology lies in the capability of the springs to represent a non-linear soil response including yielding under compressive stresses, zero response under tensions stresses as well as rocking and uplifting.

The concept of elastic elements supported by springs was firstly proposed by Emir Winkler (1867) as way to compute the stress distribution pad through a flexible medium. Subsequently, several authors had proposed other formulations such as McClelland and Focht (1958), Matlock (1970), Boulanger et al. (1999), Harden et al. (2005), Raychowdhury and Hutchinson (2008) to name a few. In the present study OpenSees (McKenna, 1997) program was used to perform the structural analyses. Raychowdhury et Hutchinson (2008) introduced a BNWF concept into OpenSees and therefore this methodology was used in the present study. They proposed the implementation of coupled springs stiffness to account for vertical and rotations displacements allowing a realistic representation of the rocking behavior of foundations. More detail regarding the parameters of the model used in this project will be provided in the next chapter.

### 2.2.3 Kinematic interaction

The input motion of a massless foundation-structure system differs from the free-field ground motion due to the kinematic interaction. Kinematic interaction occurs when the free-field waves encounter the relatively rigid material of the foundation. This induces a diffraction of seismic waves away of the foundation due to high soil-to-structure rigidity ratio. A spatially variability in ground motions within the footprint of the foundation is another source that scatter the input of energy into the system. This phenomenon is known as *base-slab averaging*. Moreover, ground motions at foundation's embedment depth differ from the motions at the free surface because of the attenuation on the soil column. Another important factor in the kinematic interaction is the natural filtering of seismic waves with wave-length about the footing length or more due to the overlapping of the wave along the footing length. (Housner 1957). Kramer and Stewart (2004) showed that this interaction introduces smaller seismic demands into the system foundation-structure. In order to keep consistency with the previous studies (Ushathan 2018), this kinematic interaction is not considered in the scope of this study.

### 2.2.4 Inertial interaction

Inertial interaction is the dynamic interaction between the structure, foundation and the surrounding soil caused by the foundation input motion. Base shears and overturning moments are developed in the structure because the acceleration of the base and transferred into the soil by the foundation. This imposes stresses and deformations in the surrounding soil and generated modification of the response in the complete system soil-foundation-structure such as period lengthening and an occurrence of additional damping mechanism (radiation and soil damping). This phenomenon is illustrated in Figure 2.32.

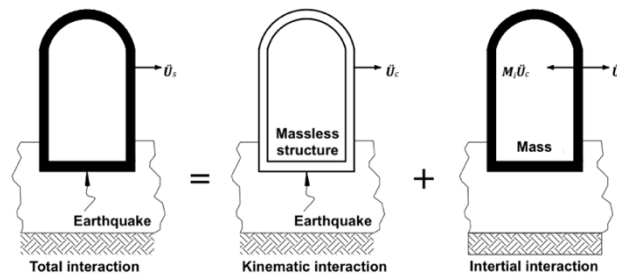


Figure 2.32 Total interaction is given as the sum of kinematic interaction and inertial interaction. Adapted from CFE (2017).

Period lengthening is a result of the flexibility of the soil. As discussed earlier, when structure-to-soil stiffness ratio is more prominent, SSI will be more significant. To illustrate this, one can consider the example of single degree of freedom structures illustrated in Figure 2.33, one with a rigid base and the other with a flexible base and estimate their fundamental periods,  $T$ .

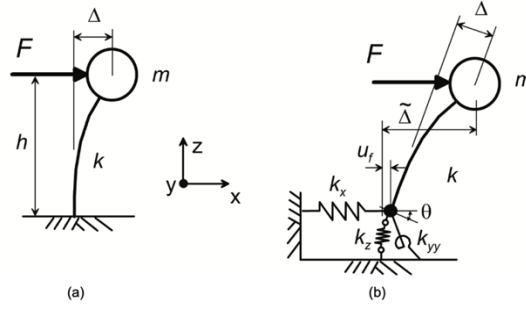


Figure 2.33 Single degree of freedom structure with rigid base (a) and with flexible base (b). Adapted from NEHRP (2012).

Following dynamic mechanics principles, the period of the structures can be determined as follows.

$$\Delta = \frac{F}{k} \quad (2.21)$$

$$\omega = \sqrt{\frac{k}{m}} \quad (2.22)$$

$$T = \frac{2\pi}{\omega} = 2\pi \sqrt{\frac{m}{k}} \quad (2.23)$$

$$T^2 = (2\pi)^2 \frac{m}{F/\Delta} = (2\pi)^2 \frac{m\Delta}{F} \quad (2.24)$$

The ratio of the periods of the structure with flexible and fixed base can be determined as follows.

$$\tilde{\Delta} = \frac{F}{k} + u_f + \theta \cdot h \quad (2.25)$$

$$\tilde{\Delta} = \frac{F}{k} + \frac{F}{k_x} + \left( \frac{F \cdot h}{k_{yy}} \right) \quad (2.26)$$

$$\tilde{T}^2 = (2\pi)^2 \frac{m\tilde{\Delta}}{F} = (2\pi)^2 \cdot m \cdot \left( \frac{1}{k} + \frac{1}{k_x} + \frac{h^2}{k_{yy}} \right) \quad (2.27)$$

$$\left( \frac{\tilde{T}}{T} \right)^2 = \frac{k}{m} \cdot m \cdot \left( \frac{1}{k} + \frac{1}{k_x} + \frac{h^2}{k_{yy}} \right) \quad (2.28)$$

$$\frac{\tilde{T}}{T} = \sqrt{1 + \frac{k}{k_x} + \frac{kh^2}{k_{yy}}} \quad (2.29)$$

where:

$k$  = vertical stiffness of the soil

$k_x$  = lateral stiffness of the soil

$k_{yy}$  = rotation stiffness of the soil

$m$  = mass of the system

$\tilde{T}$  = flexible base period

$T$  = fixed base period

Equation 2.27 represents the period lengthening ratio. It can be seen that when  $k_x$  and  $k_{yy}$  are greater than 0 (i.e. SSI implemented), the ratio will be greater than 1 (lengthening of the period occurs).

In addition to period lengthening, inertial interaction causes additional source of damping, namely (i) *hysteretic damping* and (ii) *radiation damping*. Hysteretic damping occurs by the direct deformation and restoration of the materials involved in the interaction. On the other hand, radiation damping results from the physical rocking of the footing on the soil, radiating away interface waves from the foundation and carrying energy away from the system.

Many damping models have been proposed by different authors. The classical solution for frequency dependence of foundation in damping terms presented by Veletsos et Nair (1975) provides a comprehensive methodology to consider damping increasing phenomenon and is widely used.

$$\beta_0 = \beta_f + \frac{\beta_i}{\left( \frac{\tilde{T}}{T} \right)^n} \quad (2.30)$$

where:

$\beta_o$  = flexible base system damping

$\beta_f$  = foundation damping

$\beta_i$  = assumed structural damping with fixed base

$n$  = exponential parameter taken as 3 for linearly viscous structural damping. Taken as 2 for all other cases.

Inertial interaction can be described by impedance functions which are mathematical expressions used to describe the stiffness and damping characteristics of the SSI. Impedance functions are constitutive law parameters that describe the behaviour of the inertial interaction between foundation and soil. They take into consideration the geometry and stiffness of the foundation as well as soil shear modulus and the Poisson ratio. As they describe the soil response at material level, they can be incorporated into analytical elements to introduce explicitly SSI into analysis models. Equations proposed by Gazetas and Stokoe (1991) and presented in Figure 2.34, for impedance functions are adopted for many researchers as they permit easy inclusion of different parameters such as degrees of freedom involved in the interaction, embedment depths, damping and footing geometry, etc. These equations were used in the scope of the present study.

Impedance functions permits to estimate the static foundation stiffness  $K$  for three translational and three rotational degrees of freedom for a rigid rectangular footing located at the ground surface. An embedment factor  $e$  is then added to increase footing stiffness according to the foundation depth.

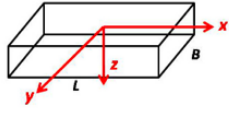
| Stiffness                                                    | Equation                                                                                                                                                                  |  |
|--------------------------------------------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------|
|                                                              | Surface Stiffness                                                                                                                                                         |                                                                                     |
| Vertical Translation                                         | $K_z' = \frac{GL}{1-\nu} \left[ 0.73 + 1.54 \left( \frac{B}{L} \right)^{0.75} \right]$                                                                                    |                                                                                     |
| Horizontal Translation (toward long side)                    | $K_y' = \frac{GL}{2-\nu} \left[ 2 + 2.5 \left( \frac{B}{L} \right)^{0.85} \right]$                                                                                        |                                                                                     |
| Horizontal Translation (toward short side)                   | $K_x' = \frac{GL}{2-\nu} \left[ 2 + 2.5 \left( \frac{B}{L} \right)^{0.85} \right] + \frac{GL}{0.75-\nu} \left[ 0.1 \left( 1 - \frac{B}{L} \right) \right]$                |                                                                                     |
| Rotation about x-axis                                        | $K_{\theta_x}' = \frac{G}{1-\nu} I_x^{0.75} \left( \frac{L}{B} \right)^{0.25} \left( 2.4 + 0.5 \frac{B}{L} \right)$                                                       |                                                                                     |
| Rotation about y-axis                                        | $K_{\theta_y}' = \frac{G}{1-\nu} I_y^{0.75} \left[ 3 \left( \frac{L}{B} \right)^{0.15} \right]$                                                                           |                                                                                     |
|                                                              | Stiffness Embedment Factors                                                                                                                                               |                                                                                     |
| Embedment Factor, Vertical Translation                       | $e_z = \left[ 1 + 0.095 \frac{D_f}{B} \left( 1 + 1.3 \frac{B}{L} \right) \right] \left[ 1 + 0.2 \left( \frac{2L+2B}{LB} H \right)^{0.67} \right]$                         |                                                                                     |
| Embedment Factor, Horizontal Translation (toward long side)  | $e_y = \left[ 1 + 0.15 \left( \frac{2D_f}{B} \right)^{0.5} \right] \left\{ 1 + 0.52 \left[ \frac{\left( D_f - \frac{H}{2} \right) 16(L+B)H}{BL^2} \right]^{0.4} \right\}$ |                                                                                     |
| Embedment Factor, Horizontal Translation (toward short side) | $e_x = \left[ 1 + 0.15 \left( \frac{2D_f}{L} \right)^{0.5} \right] \left\{ 1 + 0.52 \left[ \frac{\left( D_f - \frac{H}{2} \right) 16(L+B)H}{LB^2} \right]^{0.4} \right\}$ |                                                                                     |
| Embedment Factor, Rotation about x axis                      | $e_{\theta_x} = 1 + 2.52 \frac{H}{B} \left( 1 + \frac{2H}{B} \left( \frac{d}{D_f} \right)^{-0.2} \left( \frac{B}{L} \right)^{0.5} \right)$                                |                                                                                     |
| Embedment Factor, Rotation about y axis                      | $e_{\theta_y} = 1 + 0.92 \left( \frac{2H}{L} \right)^{0.60} \left( 1.5 + \left( \frac{2H}{L} \right)^{1.9} \left( \frac{H}{D_f} \right)^{-0.60} \right)$                  |                                                                                     |

Figure 2.34 Impedance equation from Gazetas and Stokoe (1991). Adapted from Raychowdhury and Hutchinson (2008).

## **CHAPTER 3      PROTOTYPE BUILDING DESIGN**

In this section, the design of the frame in the prototype buildings is presented. 3- and 6-storey building were designed for Montreal and Vancouver following NBCC 2020 and CSA S16-19 provisions. Site Classes C and E were considered to investigate the influence of soil rigidity. The building layout was adopted from Muruganathan (2018), to provide basis for results comparison. Muruganathan (2018) carried out an exploratory study for 3-storey frames located in Vancouver and Montreal and designed according to NBCC 2015 and CSA S16-14.

### **3.1 Building definition**

The studied building illustrated in Figure 3.1, is a typical office building, 32.5 m wide and 54.5 m long, with 250 mm slab overhang on the whole perimeter of the building. Its longest side is positioned in East-West direction. The typical storey height is 4 m while the first storey is 4.5 m. The overall height of the building is  $h_n=24.5$  m. A typical slab consists of a steel-deck with 76 mm ribs and 65mm concrete thickness above the ribs. It is assumed the slab acts as a rigid diaphragm. The roof slab does not have concrete topping it but includes a roof membrane.

A typical layout of the building for Montreal and Vancouver is shown in Figure 3.2 (a) and (b), respectively. In N-S direction the building is braced by X-type compression-tension moderately ductile CBFs. Two inner CBFs are placed symmetrically. Since Western Canada has a considerably stronger seismic activity than Easter Canada, for Vancouver building two additional CBFs are located on the perimeter in the N-S direction. In the E-W direction, lateral load is resisted by perimeter moment-resisting frames.

Live load was taken equal to 2.4 kPa as per NBCC 2020 for office buildings while the dead load was determined from the slap geometry considering the weight of the common building finishes. Partitions load was taken equal to 1 kPa. Brick perimeter wall were considered for Montreal with a weight of 1.5 kPa while for Vancouver a curtain wall was chosen with a weight of 1 kPa. A reduction of 0.5 kPa for partitions was considerate to determine the seismic weight of the building. Importance factor equal to normal ( $I_E=1$ ) was considered. Snow and rain loading were calculated for both locations as per NBCC 2020. A summary of vertical loads is given in Table 3.1.

Table 3.1 Vertical design loads

| Load pattern          | Symbol | Montreal | Vancouver |
|-----------------------|--------|----------|-----------|
|                       |        | kPa      | kPa       |
| Dead (typical storey) | D      | 4.60     | 4.60      |
| Dead (roof)           | D      | 1.35     | 1.35      |
| Live                  | L      | 2.40     | 2.40      |
| Snow / Rain           | S      | 2.48     | 1.64      |
| Cladding              | D      | 1.50     | 1.00      |

All building elements were designed for relevant load combinations defined by NBCC 2020 listed below.

Dead load  $\rightarrow 1.4 D$

Live load  $\rightarrow 1.25 D + 1.5 L + 1.0 S$

Snow/rain loads  $\rightarrow 1.25 D + 1.5 S + 1.0 L$

Lateral wind  $\rightarrow 1.25 D + 1.4 W + 0.5 L + 0.5 S$

Lateral earthquake  $\rightarrow D + E$

Lateral earthquake  $\rightarrow D + E + 0.5 L + 0.25 S$

Beams and columns were checked to ensure resistance for vertical loads cases. All SRFS components (beams, columns and braces) were checked to meet all seismic design requirements as well. In addition to seismic design, the building was also checked for wind loads. It was found that wind did not govern lateral design in any case.

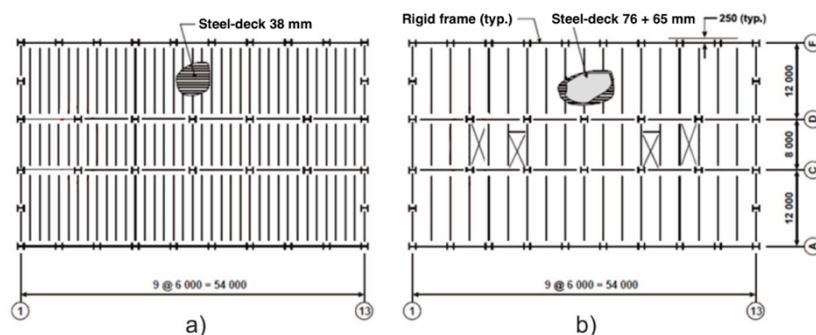


Figure 3.1 Building layout. Adapted from Tremblay 2017 and Murugananthan (2018).

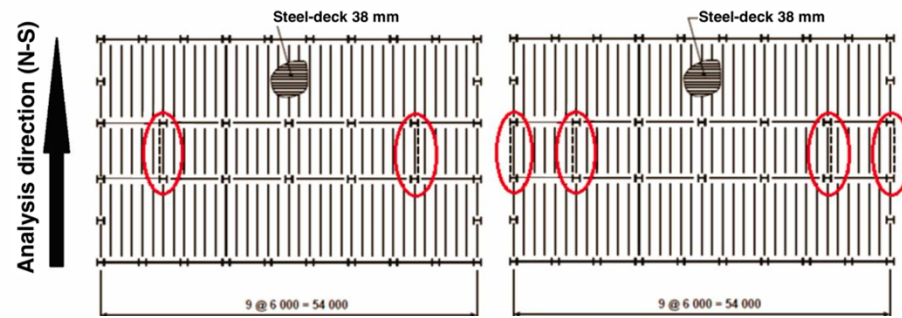


Figure 3.2 Location of SFRS for Montreal Scenarios (a) and for Vancouver scenarios (b). Adapted from Murugananathan (2018).

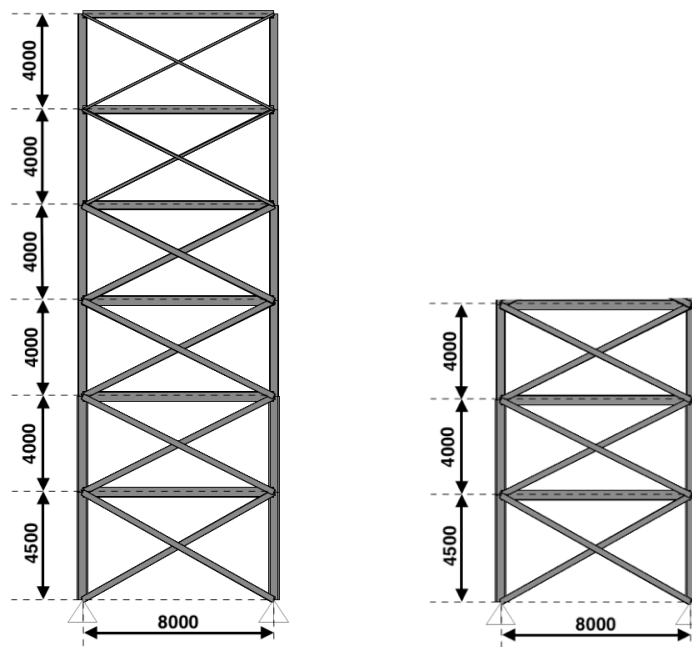


Figure 3.3 Elevation view of X-type CBF.

### 3.2 Design example

Typical procedure of CBFs design is presented on an example of 6-storey building in Vancouver on Site Class C.

Similar procedures were followed for all studied scenarios. Excel tools were developed to perform and automatize the calculations. Even though the building were regular, a dynamic spectral analysis has been chosen to capture better the dynamic behaviour of the building, and design office practices.

Seismic demands for buildings depend on several factors associated with environmental conditions and structural inherent characteristics. Environmental conditions are local seismic hazard and site situation (soil amplification or attenuation of seismic waves). Structural inherent characteristic that influence the seismic demand is primarily structural period which depends on stiffness and structural mass. Because this intrinsic dependence of the structural period and the seismic demands, the design becomes an iterative process.

A preliminary calculation with a trial period is made in order to estimate the seismic forces. Then the design of the structural elements is performed to sustain this demand. However, it may happen that the selected profiles modify the stiffness of the structure resulting in modified structural period, which, in turn, imposes another iteration. In this example only the final iteration is showed.

The seismic weight of the building was calculated considering the weight of the building and 25% of the snow loading.

Table 3.1 Structure mass

| Storey | h (m) | $h_i$ (m) | $W_i$ (kN) |
|--------|-------|-----------|------------|
| 6      | 4.0   | 24.5      | 3465       |
| 5      | 4.0   | 20.5      | 7958       |
| 4      | 4.0   | 16.5      | 7958       |
| 3      | 4.0   | 12.5      | 7958       |
| 2      | 4.0   | 8.5       | 7958       |
| 1      | 4.5   | 4.5       | 8002       |
| Total  | --    | --        | 43300      |

Next, the design spectrum was established for Vancouver's City Hall location. In the NBC 2020 a user can input the actual shear wave velocity,  $V_{S30}$ , at the design site to calculate the modified hazard values for site condition. In this study, the traditional approach is taken based on the site classification from Class A to F, to evaluate, among other, the impact of such procedure on design and conclude on benefits to conduct site-specific geotechnical studied. Design is done for reference Class C site (very dense soil and soft rock), was taken in this example considering a site designation  $X_C$ . This designation corresponds to the lowest  $V_{S30}$  given in function of the  $V_{S30}$  characterizing the site Class C (i.e. 360 m/s<sup>2</sup>). Calculated hazard values are given in Table 3.2.

Table 3.2 Hazard values in Vancouver's City Hall for site Class C.

| T (s) | S(T,C) (g) |
|-------|------------|
| 0.1   | 1.090      |
| 0.2   | 1.090      |
| 0.5   | 0.876      |
| 1     | 0.508      |
| 2     | 0.309      |
| 5     | 0.088      |
| 10    | 0.037      |
| PGA   | 0.475      |
| PGV   | 0.529      |

Subsequently the Seismic Category of the building can be defined considering the hazard values at 0.2s and 1s as shown in Table 3.3. The most severe between the two obtained values is considered to be a final Seismic Category for the building.

Table 3.3 Seismic Category for Site Class C, Vancouver scenario.

|              |                            |     |
|--------------|----------------------------|-----|
| Short period | $I_E S(0.2) = 1.09 > 0.75$ | SC4 |
| Long period  | $I_E S(1.0) = 0.51 > 0.30$ | SC4 |

In the next step, an appropriate seismic force resisting system is selected. In this study the building is braced by moderately ductile CBFs. In the practice, the decision regarding which SFRS will be employed could be complex. Numerous factors play a role in the final decision such as architectural concepts, construction logistics, supply chain and project management to name a few.

As discussed earlier, the period of the structure should be determined to calculate the design spectral acceleration. NBC proposes an empirical method to estimate the period ( $T_{emp}$ ) for a selected SFRS with a given height ( $h_n$ ). A maximum period ( $T_{max}$ ) limits the design period in order to provide a minimum lateral resistance. Finally, after several design iterations, the actual period of the structure, ( $T_1$ ), can be estimated using available mechanical methods (example modal analysis) for each direction under study and compared with  $T_{max}$ . The period used for design ( $T_a$ ) is then determined in function of  $T_1$ , without exceeding the maximum limit ( $T_{max}$ ). Period calculation for the example building is given in Table 3.4.

Table 3.4 Calculation of design period  $T_a$  for six-storey, site Class C, Vancouver scenario.

| Period                 | Value | Reference article NBC 2020 |
|------------------------|-------|----------------------------|
| $T_{emp} = 0.025h_n$   | 0.613 | 4.1.8.11.3 from a to c     |
| $T_{max} = 2.0T_{emp}$ | 1.225 | 4.1.8.11.3.d               |
| $T_1$                  | 0.913 | From modal analysis        |
| $T_a$                  | 0.913 | 4.1.8.11. (3)              |

Once the design period is estimated ( $T_a$ ), the design spectral acceleration  $S(T_a)$  can be calculated, using interpolation. NBC 2020 allows the use of logarithmic interpolation resulting in slightly smaller design acceleration. Design acceleration is finally checked against the maximum and minimum spectral accelerations as described in Section 2.1.2 and the final value of  $S(T_a)$  is determined. Note that  $S(T_a)$  has to be corrected by  $M_v$  factor that accounts for the effect of higher modes. For the example structure  $M_v$  is equal to 1.

Table 3.5 Design spectral acceleration calculations for six-storey, site Class C, Vancouver scenario.

| Parameter                                   | Spectral acceleration<br>(g) |       | Reference article<br>NBC 2020 |
|---------------------------------------------|------------------------------|-------|-------------------------------|
| Interpolated spectral response acceleration | $S(T_a)$                     | 0.546 | 4.1.8.11.2                    |
| Maximum spectral acceleration               | $2/3 S(0.2)$                 | 0.727 | 4.1.8.11.2.c                  |
| Maximum spectral acceleration               | $S(0.5)$                     | 0.876 | 4.1.8.11.2.c                  |
| Minimum spectral acceleration               | $S(2)$                       | 0.309 | 4.1.8.11.2.a & b              |
| Design elastic acceleration                 | $S(T_a)$                     | 0.546 | --                            |

Once the spectral acceleration is determined, the equivalent static base shear can be estimated ( $V$ ). The calculations are summarized in Table 3.6. This base shear will be used later to calibrate the dynamic base shear and calculate the accidental torsion moments.

Table 3.6 Calculations for static equivalent base shear for six-storey, site Class C, Vancouver scenario.

| Parameter                                       |                                                      | Value |    | Reference article<br>NBC 2020 |
|-------------------------------------------------|------------------------------------------------------|-------|----|-------------------------------|
| Higher mode effect on base shear factor         | $M_v$                                                | 1.00  | -- | Table 4.1.8.11.               |
| Seismic weight of the structure                 | $W$                                                  | 43300 | kN | 4.1.8.2.                      |
| Importance factor                               | $I_E$                                                | 1.00  | -- | 4.1.8.5.                      |
| Reduction factor for ductility and overstrength | $R_d R_o$                                            | 3.90  | -- | 4.1.8.9.                      |
| Elastic static base shear                       | $V_e = S(T_a) \cdot M_v \cdot I_E \cdot W$           | 23635 | kN | --                            |
| Inelastic static base shear                     | $V = S(T_a) \cdot M_v \cdot I_E \cdot W / (R_d R_o)$ | 6060  | kN | 4.1.8.11.2                    |

In the next step, a spectral analysis is performed using SAP2000 to evaluate the seismic effects of seismic loads on the structure. After several iterations, the final section profiles have been chosen for all frame elements complying to all the code requirements.

The base shear computed by the analysis  $V_{ed, SAP2000}$  is then scaled as prescribed by NBCC2020. The inelastic dynamic base shear,  $V_d$ , divided by  $R_d R_o$ , the product of ductility and overstrength related factors and compared to 80% of  $V$  calculated by the equivalent static force procedure. If

this threshold is not met,  $V_d$  should be scaled up to reach that value. In the example  $V_d$  was higher than  $0.8 V$  and no scaling up was necessary (i.e. the calibration factor is equal to 1).

Table 3.7 Summary of the results of the spectral analysis for six-storey, site Class C, Vancouver scenario.

|                                                       |                                          |        |
|-------------------------------------------------------|------------------------------------------|--------|
| Multiplier factor 1                                   | $MF1 = 2/3 \cdot S(0.2) / S(T_a) \leq 1$ | 1.00   |
| Multiplier factor 2                                   | $MF2 = S(0.5) / S(T_a) \leq 1$           | 1.00   |
| Elastic dynamic base shear from dynamic analysis (kN) | $V_{ed, SAP2000}$                        | 20098  |
| Elastic dynamic base shear (kN)                       | $V_{ed} = MF \cdot V_{ed, SAP2000}$      | 20098  |
| Inelastic dynamic base shear (kN)                     | $V_d = V_{ed} / (R_d R_o)$               | 5153   |
| Dynamic - static base shear ratio                     | $V_d / V$                                | 85.03% |
| Minimum dynamic base shear                            | $(V_d / V)_{min}$                        | 80.00% |
| Calibration factor                                    | $S_c$                                    | 1.00   |

The effect of notional loads and P- $\Delta$  effect need to be added to determine the final lateral force demand. It should be noted that  $V_d$  has been calculated for the complete building. To determine the force attributed to a single frame,  $V_d$  was divided by the number of frames and increased for the corresponding portion of the accidental torsion. In this example the  $0.32 V_d$  is attributed to the loaded frames ( $0.25 V_d$  without accidental torsion +  $0.07$  due to accidental torsion). The summary of the final design loads is given in Table 3.8.

Table 3.8 Seismic lateral demand for six-storey, site Class C, Vancouver scenario.

| Storey | Complete building storey shear |                    |                    | Percentage of shear per frame |                    | Shear per frame | Brace force |
|--------|--------------------------------|--------------------|--------------------|-------------------------------|--------------------|-----------------|-------------|
|        | Notional Loads (kN)            | Seismic Loads (kN) | P- $\Delta$ factor | Notional Loads (kN)           | Seismic Loads (kN) | $V_f$ (kN)      | $C_f$ (kN)  |
| 6      | 17                             | 1008               | 1.02               | 0.25                          | 0.32               | 332             | 186         |
| 5      | 66                             | 2397               | 1.03               | 0.25                          | 0.32               | 803             | 449         |
| 4      | 113                            | 3353               | 1.03               | 0.25                          | 0.32               | 1134            | 634         |
| 3      | 161                            | 4144               | 1.03               | 0.25                          | 0.32               | 1414            | 790         |
| 2      | 209                            | 4625               | 1.04               | 0.25                          | 0.32               | 1593            | 890         |
| 1      | 257                            | 5227               | 1.05               | 0.25                          | 0.32               | 1816            | 1042        |

Once the brace axial force,  $C_f$ , is determined, diagonals' profiles can be selected. CSA S16-19 prescribe the strength and ductility related criteria to ensure a stable inelastic response for the structural elements, allow them to develop significant inelastic deformation and thus dissipate seismic energy. As explained previously in Chapter 2, effective length of the compressed diagonals was calculated considering that they are laterally supported by the tensed diagonals and that the connection dimensions reduce their length. Taken these two factors into consideration the effective length,  $kL$ , was assumed equal to 0.45 the complete length of the diagonals. Wall-thickness-to-depth ratio was chosen to avoid local buckling, considered as a brittle mode of failure. To calculate the brace factored resistance,  $F_y$ , is taken equal to 345 MPa as specified by S16-19 for HSS made of G40.20 steel. Finally, as seismic category was determined as SC4, the slenderness ratio was limited to a minimum of 70 as required to assure brace buckling and seismic energy dissipation.

Once the braces are chosen, their probable capacity is calculated in tension and compression to determine design forces for beam and columns following the capacity design principles making sure that in any case the probable axial load for columns does not overpass the correspondent axial load for an elastic earthquake, i.e. with a  $R_d R_o$  equal to 1.3. For columns, vertical components of these loads are combined with tributary vertical loading induced by dead load loads, reduced live loads and snow loads. For column the load combination  $D+E+0.5L+0.25S$  was critical. In addition to vertical force, a flexural moment equal to 0.2 times the column plastic moment was considered in design. Columns were then checked as beam-column in accordance with S16-19 interaction equations for cross-section yielding (A), flexural buckling (B) and lateral-torsional buckling (C). The summary of relevant calculation for column design is provided in Table 3.9.

Table 3.9 Summary of column design for six-storey, site Class C, Vancouver scenario.

| Storey | $C_f$<br>(kN) | Profile  | Class | $kL/r$ | Interaction<br>A | Interaction<br>B | Interaction<br>C |
|--------|---------------|----------|-------|--------|------------------|------------------|------------------|
| 6      | 552           | W360X91  | 1     | 64     | 0.35             | 0.35             | 0.22             |
| 5      | 1881          | W360X91  | 1     | 64     | 0.94             | 0.94             | 0.22             |
| 4      | 3520          | W360X196 | 1     | 42     | 0.70             | 0.70             | 0.22             |
| 3      | 5547          | W360X196 | 1     | 42     | 0.96             | 0.96             | 0.22             |
| 2      | 7633          | W360X382 | 1     | 38     | 0.75             | 0.75             | 0.22             |
| 1      | 10195         | W360X382 | 1     | 43     | 0.92             | 0.92             | 0.22             |

Similarly, beams are designed for the combination of tributary vertical loads and capacity forces from braces. Lateral bracing along the beam are provided in site for this objective. Beams are considered pinned connected to the columns and fully laterally braced out-of-plane. Summary of beam design is given in Table 3.10.

Table 3.10 Summary of beam's design for six-storey, site Class C, Vancouver scenario.

| Storey | E                      | D+0.5L+0.25S             | 1.25D+1.5L+1S <sup>1</sup> | Beam design |       |                                          |
|--------|------------------------|--------------------------|----------------------------|-------------|-------|------------------------------------------|
|        | C <sub>E</sub><br>(kN) | M <sub>r</sub><br>(kN-m) | M <sub>r</sub><br>(kN-m)   | Section     | Class | Critical<br>Demand-to-reistance<br>ratio |
| 6      | 381                    | 4.92                     | 29.52                      | W310X28.3   | 1     | 0.95                                     |
| 5      | 911                    | 110.4                    | 138                        | W310X67     | 1     | 0.94                                     |
| 4      | 1228                   | 28.8                     | 86.4                       | W360X72     | 1     | 0.95                                     |
| 3      | 1586                   | 139.2                    | 224.4                      | W360X79     | 1     | 0.99                                     |
| 2      | 1746                   | 139.2                    | 224.4                      | W360X91     | 1     | 0.91                                     |
| 1      | 1930                   | 139.2                    | 224.4                      | W360X91     | 1     | 0.99                                     |

<sup>1</sup> For roof beams, 1.25D+1L+1.5S may represent a severe condition and it shall be investigated.

### 3.3 Building design

Following the procedure illustrates in the previous sections, frames were designed for all studied scenarios. Tables 3.11 to 3.14 present the profiles selected for all frame members.

Table 3.11 Six-storey buildings: brace sections

| Location    | Montreal      |               | Vancouver     |               |
|-------------|---------------|---------------|---------------|---------------|
| Site Class  | E             | C             | E             | C             |
| Design code | NBC2020       | NBC2020       | NBC2020       | NBC2020       |
| Storey 6    | HS102X102X7.9 | HS102X102X6.4 | HS102X102X7.9 | HS102X102X6.4 |
| Storey 5    | HS127X127X9.5 | HS127X127X6.4 | HS127X127X13  | HS114X114X7.9 |
| Storey 4    | HS152X152X9.5 | HS127X127X6.4 | HS254X127X13  | HS127X127X9.5 |
| Storey 3    | HS152X152X9.5 | HS127X127X7.9 | HS254X127X13  | HS127X127X13  |
| Storey 2    | HS152X152X13  | HS127X127X9.5 | HS254X127X16  | HS178X127X9.5 |
| Storey 1    | HS203X152X13  | HS152X152X7.9 | HS254X152X16  | HS178X127X13  |
| Weight (kg) | 4541          | 2899          | 6860          | 3824          |

Table 3.12 Six-storey buildings: column sections

| Location    | Montreal |          | Vancouver |          |
|-------------|----------|----------|-----------|----------|
| Site Class  | E        | C        | E         | C        |
| Design code | NBC2020  | NBC2020  | NBC2020   | NBC2020  |
| Storey 6    | W360X101 | W360X91  | W360X122  | W360X91  |
| Storey 5    | W360X101 | W360X91  | W360X122  | W360X91  |
| Storey 4    | W360X216 | W360X179 | W360X287  | W360X196 |
| Storey 3    | W360X216 | W360X179 | W360X287  | W360X196 |
| Storey 2    | W360X421 | W360X314 | W360X551  | W360X382 |
| Storey 1    | W360X421 | W360X314 | W360X551  | W360X382 |
| Weight (kg) | 12063    | 9645     | 15923     | 11095    |

Table 3.13 Six-storey buildings: beam sections

| Location    | Montreal  |           | Vancouver |           |
|-------------|-----------|-----------|-----------|-----------|
| Site Class  | E         | C         | E         | C         |
| Design code | NBC2020   | NBC2020   | NBC2020   | NBC2020   |
| Storey 6    | W310X32.7 | W310X28.3 | W310X32.7 | W310X28.3 |
| Storey 5    | W360X72   | W310X67   | W360X79   | W310X67   |
| Storey 4    | W360X79   | W310X67   | W360X110  | W360X72   |
| Storey 3    | W360X91   | W360X72   | W410X114  | W360X79   |
| Storey 2    | W360X91   | W360X72   | W410X132  | W360X91   |
| Storey 1    | W360X110  | W360X79   | W410X132  | W360X91   |
| Weight (kg) | 3973      | 3064      | 4821      | 3406      |

Table 3.14 Three-storey buildings: brace sections

| Location    | Montreal      |               | Vancouver     |               |
|-------------|---------------|---------------|---------------|---------------|
| Site Class  | E             | C             | E             | C             |
| Design code | NBC2020       | NBC2020       | NBC2020       | NBC2020       |
| Storey 3    | HS102X102X9.5 | HS102X102X6.4 | HS102x102x8.0 | HS102x102x6.4 |
| Storey 2    | HS127X127X13  | HS114X114X9.5 | HS127x127x13  | HS127x127x8.0 |
| Storey 1    | HS152X152X13  | HS127X127X13  | HS152x152x13  | HS152x152x8.0 |
| Weight (kg) | 2180          | 1630          | 2115          | 1472          |

Table 3.15 Three-storey buildings: column sections

| Location    | Montreal |          | Vancouver |          |
|-------------|----------|----------|-----------|----------|
| Site Class  | E        | C        | E         | C        |
| Design code | NBC2020  | NBC2020  | NBC2020   | NBC2020  |
| Storey 3    | W310X226 | W310X179 | W360X196  | W310X158 |
| Storey 2    | W310X226 | W310X179 | W360X196  | W310X158 |
| Storey 1    | W310X226 | W310X179 | W360X196  | W310X158 |
| Weight (kg) | 5651     | 4474     | 4906      | 3944     |

Table 3.16 Three-storey buildings: beam sections

| Location    | Montreal  |           | Vancouver |           |
|-------------|-----------|-----------|-----------|-----------|
| Site Class  | E         | C         | E         | C         |
| Design code | NBC2020   | NBC2020   | NBC2020   | NBC2020   |
| Storey 3    | W250X32.7 | W250X28.4 | W250X28.4 | W250X28.4 |
| Storey 2    | W360X79   | W310X74   | W310X79   | W310X67   |
| Storey 1    | W360X91   | W360X79   | W360X91   | W310X79   |
| Weight (kg) | 1619      | 1454      | 1584      | 1393      |

### 3.4 Gusset plate design

Once the section for braces, columns and beams had been selected, the connection of the diagonals to the rest of the structure were designed. According to capacity design philosophy, connection must sustain the probable forces developed by the DCE. For these reasons each limit state of the connections was checked under  $T_{prob}$  and  $C_{prob}$  brace actions.

Procedure for gusset design with an example of principal calculation is presented below.

#### Scenario: 6S-NBC20-VCR-C

- Location: Storey 2: Lower position
- Brace profile: HS178X127X9.5
  - $T_{prob}$ : 2398 kN
  - $C_{prob}$ : 1341 kN
- Column W360X382
- Beams W360X91

The slotted length,  $L_w$ , was first established. As common practice,  $L_w$  is taken as twice the width of the brace in order to provide a dimension large enough to reduce the shear lag effect. A trial  $L_w$  should be validated to ensure an adequate welding capacity and a tear-out resistance greater than  $T_{prob}$ .

Once the  $L_w$  is set, the resistance of the gusset plate can be evaluated. A trial thickness is selected and the Whitmore effective width is calculated by using a 30-degrees projection from the end of the welds placed along  $L_w$  up to the parallel projection to the edge of the brace. Then, tensile and compressive resistance of the gusset-plate cross section is checked. To calculate flexural buckling resistance, the effective buckling length is determined following the recommendations by Roeder et al. (2011) who proposes to take the average distance of  $L_1$ ,  $L_2$ ,  $L_3$  illustrated in Figure 3.4.

Finally, the complete geometry of the plate is established by positioning the Whitmore width in such manner that a plastic hinge can be developed as discussed in Chapter 2. Complementary calculations such as slot reinforcement and plate/beam or plate/column interface stresses need to be validated as well.

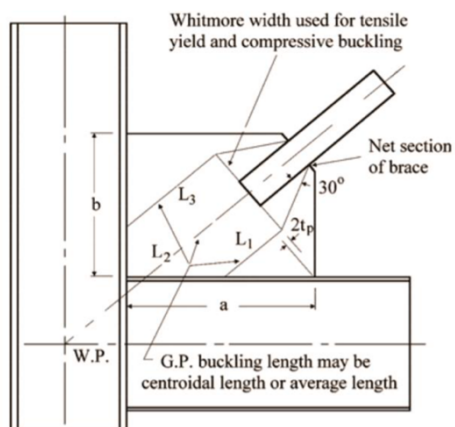


Figure 3.4 Buckling length L1, L2, L3 and Whitmore width. Adapted from Roeder et al. (2011).

Table 3.17 Connection welds resistance for six-storey, site Class C, Vancouver scenario.

|                                         |                                                         |       |                 |
|-----------------------------------------|---------------------------------------------------------|-------|-----------------|
| Electrode tensile strength              | $X_u$                                                   | 490   | MPa             |
| Fillet size                             | D                                                       | 12    | mm              |
| Slot length                             | $L_w$                                                   | 355   | mm              |
| Total length of welds                   | $L = 4 \cdot (L_w)$                                     | 1420  | mm              |
| Effective area of failure of th welds   | $A_w = 0.7071 \cdot D \cdot L$                          | 12049 | mm <sup>2</sup> |
| Inclination of load                     | $\square$                                               | 0     | degrees         |
| Increase factor due to weld inclination | $f_s = (1 + 0.5 (\sin \theta)^{1.5})$                   | 1.00  | --              |
| Phi factor (rupture of welds)           | $\phi_w$                                                | 0.67  | --              |
| Welds factored strength                 | $V_r = \phi_w \cdot 0.67 \cdot A_w \cdot X_u \cdot f_s$ | 2650  | kN              |
| Factored load                           | $V_f = T_{\text{prob}}$                                 | 2398  | kN              |
| Demand / capacity ratio                 | D/C                                                     | 0.90  | --              |

Table 3.18 Connection tear-out resistance for six-storey, site Class C, Vancouver scenario.

|                                       |                                                         |       |                 |
|---------------------------------------|---------------------------------------------------------|-------|-----------------|
| Gross section area subjected to shear | $A_{gv} = 4 \cdot L_w \cdot t_{brace}$                  | 13526 | mm <sup>2</sup> |
| Brace yield strength                  | $F_y$                                                   | 345   | MPa             |
| Brace tensile strength                | $F_u$                                                   | 450   | MPa             |
| Phi factor                            | $\phi_u$                                                | 0.75  | --              |
| Tear out factored strength            | $V_r = \phi_u [0.6 \cdot A_{gv} \cdot (F_y + F_u) / 2]$ | 2419  | kN              |
| Factored load                         | $V_f = T_{prob}$                                        | 2398  | kN              |
| Demand / capacity ratio               | $D/C = V_f / V_r$                                       | 0.99  | --              |

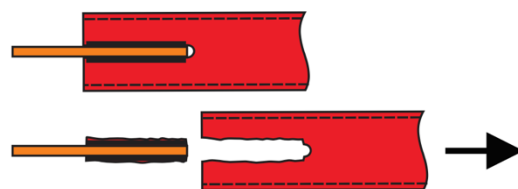


Figure 3.5 Tear-out failure. Adapted from Tremblay (2022).

Table 3.19 Connection tensile resistance for six-storey, site Class C, Vancouver scenario.

|                                           |                                 |       |                 |
|-------------------------------------------|---------------------------------|-------|-----------------|
| Whitmore effective width (30° projection) | $W_{w(30)}$                     | 588   | mm              |
| Trial gusset plate thickness              | $t_{plate}$                     | 32    | mm              |
| Cross section area at Whitmore width      | $A = W_{w(30)} \cdot t_{plate}$ | 18660 | mm <sup>2</sup> |
| Gusset plate yielding strength            | $F_y$                           | 300   | MPa             |
| Phi factor                                | $\phi$                          | 0.9   | --              |
| Factored tensile strength                 | $T_r = \phi \cdot F_y \cdot A$  | 5038  | kN              |
| Factored tensile demand                   | $T_f = T_{prob}$                | 2398  | kN              |
| Demand / capacity ratio                   | $D/C =$                         | 0.48  | --              |

Table 3.20 Connection compression resistance for six-storey, site Class C, Vancouver scenario.

|                                           |                                 |       |                 |
|-------------------------------------------|---------------------------------|-------|-----------------|
| Whitmore effective width (30° projection) | $W_{w(30)}$                     | 588   | mm              |
| Trial gusset plate thickness              | $t_{plate}$                     | 32    | mm              |
| Radius of gyration of the gusset plate    | $r = t_{plate} / \sqrt{12}$     | 9.17  | mm              |
| Cross section area at Whitmore width      | $A = W_{w(30)} \cdot t_{plate}$ | 18660 | mm <sup>2</sup> |
| Length bending buckling                   | $L = L_{ave}$                   | 540   | mm              |
| Effective length factor                   | K                               | 1.2   | --              |
| Slenderness                               | $KL/r$                          | 71    | --              |
| Stiffener yielding stress                 | $F_y$                           | 300   | MPa             |
| Elastic buckling stress                   | $F_e$                           | 126   | MPa             |
| Slenderness parameter                     | $\lambda$                       | 1.54  | --              |
| Parameter for compressive resistance      | n                               | 1.34  | --              |
| Phi factor                                | $\phi$                          | 0.9   | --              |
| Factored compressive strength             | $C_r$                           | 1726  | kN              |
| Expected compression force                | $C_f = C_{prob}$                | 1341  | kN              |
| Demand / Capacity ratio                   | $D/C =$                         | 0.78  | --              |

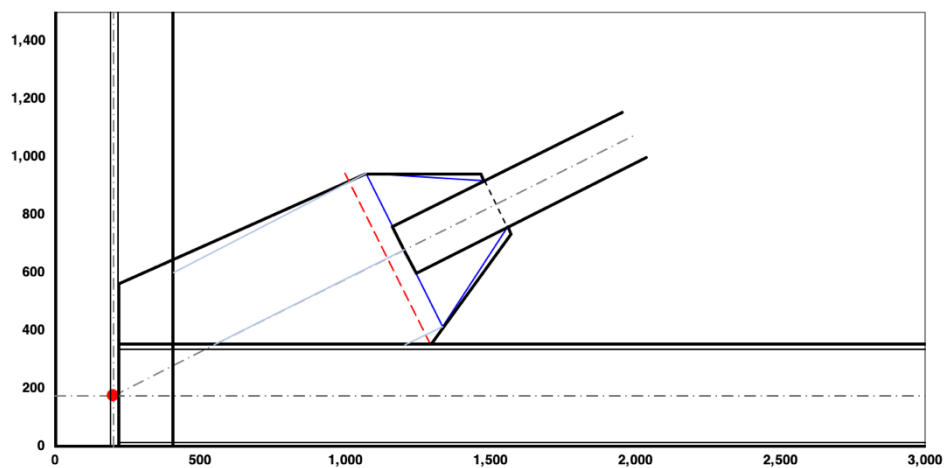


Figure 3.6 Final gusset design for 6S-NBC20-VCR-C: Storey 2-Lower position. Dark blue lines show Whitmore projection. Light blue lines show L1, L2, L3. Red dashed line marks the limit of the band to allows the plastic hinge.

### 3.5 Superstructure weight increase

All calculations discussed so far were performed in accordance with NBCC2020. To evaluate the impact of new design requirements that include the latest seismic hazard models, all studied buildings were also designed using the previous edition of NBC (NBCC 2015) and steel design standard (CSA S16-14). It has been observed that even though the design spectral acceleration increased up to 70% in some locations along a significant range of periods, the design base shear and the weight of the SFRS did not increase proportionally. In general terms, the seismic demand for Montreal locations exhibit 57% increases while in Vancouver scenarios this proportion is significantly lower with a mean of 23%. The Table 3.21 summarized the seismic base shear and the structure weight increase for each scenario.

Table 3.21 Frame weight increase

| Storey | Location | Site Class | Base shear increase | Weight increase |
|--------|----------|------------|---------------------|-----------------|
| 6S     | MTL      | C          | 56%                 | 14%             |
| 6S     | MTL      | E          | 54%                 | 28%             |
| 6S     | VCR      | C          | 25%                 | 9%              |
| 6S     | VCR      | E          | 35%                 | 27%             |
| 3S     | MTL      | C          | 52%                 | 15%             |
| 3S     | MTL      | E          | 59%                 | 21%             |
| 3S     | VCR      | C          | 19%                 | 0%              |
| 3S     | VCR      | E          | 20%                 | 8%              |

### 3.6 Foundation design

An example of foundation design is shown below. For each studied scenario, both capacity protected and not-capacity protected options were considered.

As the Canadian Foundation Engineering Manual (2006) suggests, a consistent design methodology has to be applied to super-structure and foundation thus design was performed using the *Ultimate Limit States* (ULS) approach.

A complete foundation design shall consider the resistance of the foundation itself and the capacity of the soil as well. Since the present study focuses on the SSI, which depends on the contact area between foundation and the soil and in the stresses imposed to the soil, the structural design of the footing was not performed. Complementary quick calculations were done to validate that the foundation dimensions are realistic and that a reasonable amount of flexural reinforcement is needed to ensure adequate flexural resistance. The same was done for the shear resistance of the footing. In further text, “foundation design” refers to determining the required dimensions needed to achieve acceptable bearing stress in the soil.

#### 3.6.1 Soil properties

Soil properties are chosen in such way to represent typical values of site Classes C and E. In order to remain consistent with previous studies, soil properties were directly taken from Murugananthan (2018) and validated with the industrial partner involved in the project. A resistance factor  $\phi$  equal to 0.5 is taken to reduce the ultimate capacity of the soil,  $q_{ult}$ , to its design level at factored capacity,  $q_{fact}$  as the Canadian Foundation Engineering Manual indicates. Table 3.22 list the soil properties used in design.

Table 3.22 Design soil properties

| Site Class | $q_{ult}$ (kPa) | $q_{fact}$ (kPa) | G (MPa) | $\gamma_0$ (kN/m <sup>3</sup> ) |
|------------|-----------------|------------------|---------|---------------------------------|
| C          | 3000            | 1500             | 100     | 20                              |
| E          | 400             | 200              | 20      | 20                              |

### 3.6.2 Foundation demands

From the results of the analyses and design of the super structure, demands over the foundation can be estimated. The combination of load used to estimate the seismic effect on the foundation is D + E since the most severe condition occurs when the vertical loading is the smallest.

The seismic overturning moment can be evaluated at different levels of magnitude according with the specific design objectives.

- Probable moment of the system,  $M_{pr}$ , corresponds to the moment that the CBF can develop by assuming that all braces in one direction reached their yielding probable capacity,  $T_{prob}$ , and all the other brace in the opposite direction reached their compression probable capacity,  $C_{prob}$ .
- Nominal moment of the system,  $M_n$ , is evaluated similarly to  $M_{pr}$ , with the capacity of the braces calculated with the following assumptions:
  - Use of nominal yielding stress,  $F_y$  (i.e.  $R_y=1 \rightarrow R_y F_y=F_y$ )
  - Use of resistance factor to estimate the capacity of the braces,  $\phi$ , equal to 1.
- Overturning moment with  $R_d R_o = 3.9$  equal to the overturning moment developed at the base of the CBF by the application seismic lateral load at design level with  $R_d R_o$  taken as 3.9.
- Overturning moment with different  $R_d R_o$  can be calculated by linear estimation.

Table 3.23 Foundation demands.

|                                                 |                   |        |      |
|-------------------------------------------------|-------------------|--------|------|
| Vertical force at the columns due to dead loads | D                 | 4383   | kN   |
| Probable moment of the system                   | $M_{pr}$          | 68097  | kN-m |
| Nominal moment of the system                    | $M_n$             | 51588  | kN-m |
| 75% of the nominal moment of the system         | $M_{n,75\%}$      | 38691  | kN-m |
| Overturning moment with $R_d R_o = 3.9$         | $M_{R_d R_o=3.9}$ | 29275  | kN-m |
| Overturning moment with $R_d R_o = 2$           | $M_{R_d R_o=2}$   | 57085  | kN-m |
| Overturning moment with $R_d R_o = 1$           | $M_{R_d R_o=1}$   | 114171 | kN-m |

Note that the final vertical load,  $P_f$ , includes the weight of the foundation and, if applicable, the weight of the soil above it (0.5 m in this study)

It should be noticed that since the response of the structure was evaluated structure considering SSI, the foundations were not sized according with A23.3 requirements related to foundation movements.

### 3.6.3 Capacity protected (CP) option

The foundation design is an iterative process beginning with a selection of trial dimension followed by the calculations needed to validate the choice. Design process is summarized below.

As per A23.3-19, CP foundation must be able to support the  $M_{pr}$  of the SFRS along with the vertical load,  $P_f$ . Once  $M_{pc}$  and  $P_f$  are determined, the eccentricity,  $M_{pr}/P_f$ , can be calculated. Note that in order to achieve a stable foundation, the eccentricity must lie within the footprint of the foundation. From free body diagram of the system, it can be deduced that if eccentricity is located inside the half third of the overall footing's length, no uplift will occur, otherwise, uplift happens at the opposite side of the eccentricity is located.

Soil bearing demands is then computed by dividing the vertical load by the effective contact area. Demand-to-capacity ratio should be equal or less than 1.

It should be noted that since a ULS approach is followed, a plastic stress distribution is considered as it can be seen in Figure 3.7.

Table 3.24 Trial selection of the CP foundation's dimensions.

|                                   |       |      |       |
|-----------------------------------|-------|------|-------|
| Length in direction of the moment | $h$   | 18   | m     |
| Width perpendicular to the moment | $b$   | 3.75 | m     |
| Thickness of the footing          | $t$   | 2    | m     |
| Volume                            | $V$   | 135  | $m^3$ |
| Weight of the foundation          | $W$   | 3240 | kN    |
| Weight of the soil above          | $W_s$ | 675  | kN    |

Table 3.25 CP foundation design verification.

|                                |                     |       |                |
|--------------------------------|---------------------|-------|----------------|
| Design moment                  | $M_f = M_{pr}$      | 68097 | kN/m           |
| Design axial load              | $P_f = D + W + W_s$ | 8298  | kN             |
| Eccentricity                   | $e = M_f / P_f$     | 8.21  | m              |
| Length of uniform pressure     | $h' = h - 2e$       | 1.59  | m              |
| Effective contact area         | $A = h' \cdot b$    | 5.96  | m <sup>2</sup> |
| Uniform pressure under footing | $q_{max} = P_f / A$ | 1394  | kPa            |
| Ultimate soil capacity         | $q_{fact}$          | 1500  | kPa            |
| Demand/capacity ratio          | D/C                 | 0.93  | --             |

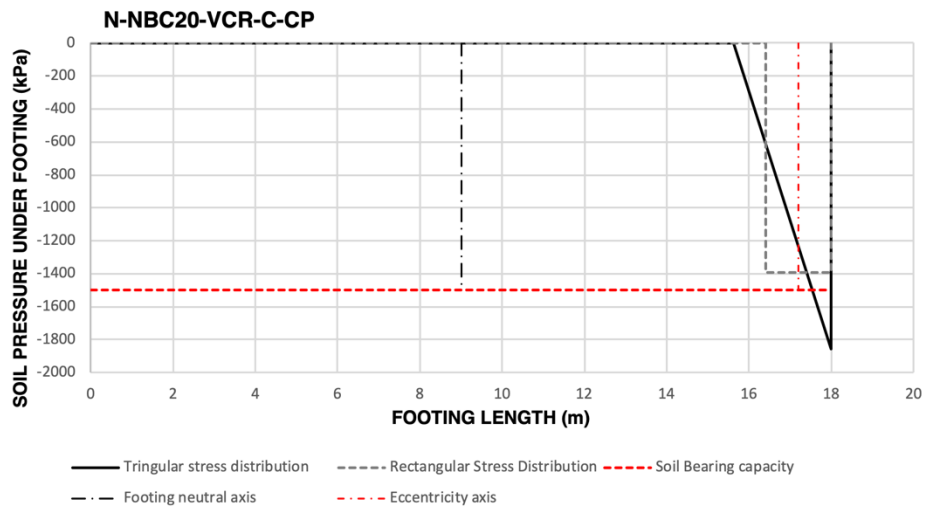


Figure 3.7 Soil's plastic and elastic stress distribution for CP foundation on site Class C.

### 3.6.4 Not-capacity protected (NCP) option

The only difference along the design process for NCP foundations, lies in the choice of the overturning moment. A23.3 code recognises the effect of SSI of limiting the input of seismic energy into the structural system, preventing the DCE of the SFRS to reach their maximum capacity. Therefore, a lesser moment is attained at the foundation. The design moment for this foundation category is set as the greater between the nominal overturning moment of the SFRS and the overturning moment corresponding to  $R_d R_o = 2$ .

As NCP results in a smaller footing dimensions, uplift of the foundation is expected to be greater leading to rotational movement of the footing and imposing additional inter-storey displacement into the superstructure. The A23.3-19 provisions address this phenomenon by analytically estimate theses lateral displacement into the structure. However, previous studies done by Murugananthan (2018) showed that inter-storey drifts remained far below design projections, yet this requirement enlarges foundations significantly. For this reason, a decision was taken to finalize foundation design without respecting the requirement of additional drift in the superstructure caused by foundation rotations.

Table 3.26 Trial selection of the NCP foundation's dimensions.

|                                   |       |      |       |
|-----------------------------------|-------|------|-------|
| Length in direction of the moment | $h$   | 16   | m     |
| Width perpendicular to the moment | $b$   | 3.75 | m     |
| Thickness of the footing          | $t$   | 2    | m     |
| Volume                            | $V$   | 120  | $m^3$ |
| Weight of the foundation          | $W$   | 2880 | kN    |
| Weight of the soil above          | $W_s$ | 600  | kN    |

Table 3.27 NCP foundation design verification.

|                                |                     |       |                |
|--------------------------------|---------------------|-------|----------------|
| Design moment                  | $M_f = M_{RdRo=2}$  | 57085 | kN/m           |
| Design axial load              | $P_f = D + W + W_s$ | 7863  | kN             |
| Eccentricity                   | $e = M_f / P_f$     | 7.26  | m              |
| Length of uniform pressure     | $h' = h - 2e$       | 1.48  | m              |
| Effective contact area         | $A = h' \cdot b$    | 5.55  | m <sup>2</sup> |
| Uniform pressure under footing | $q_{max} = P_f / A$ | 1417  | kPa            |
| Ultimate soil capacity         | $q_{fact}$          | 1500  | kPa            |
| Demand/capacity ratio          | D/C                 | 0.94  | --             |

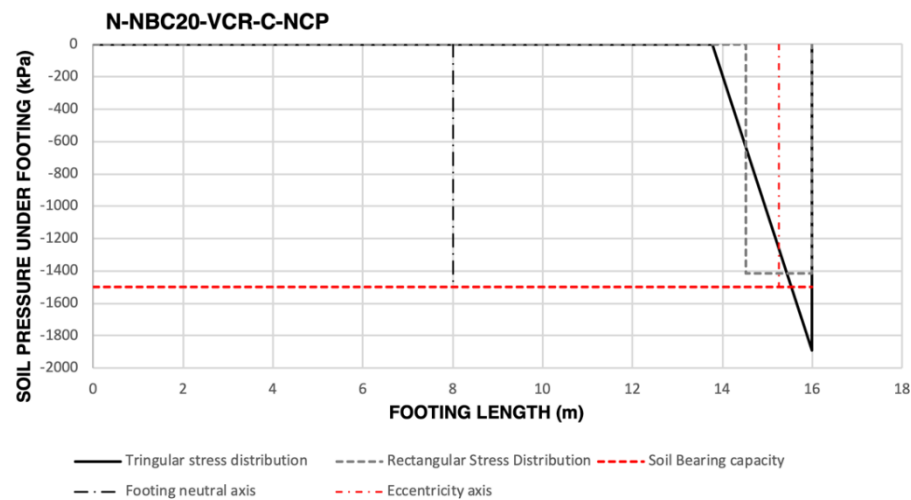


Figure 3.8 Soil's plastic and elastic stress distribution for NCP foundation on site Class C.

### 3.6.5 Differences between footing design on Site Class C and Site Class E

Due to the higher bearing capacity of the class C site compared with class E site, for all studied scenarios, foundation design was governed by the stability of the footing to prevent the overturning of the building. On the other hand, the capacity of the soil was a governing criterion for foundation design on Class E site. This observation can be validated by comparing the soil distribution stresses on both sites showed in Figure 3.8 and Figure 3.9. For Class C site, the eccentric load is placed near the edge of the foundation, while it is located near to the center for Class E site providing a larger effective contact area between the foundation and the soil.

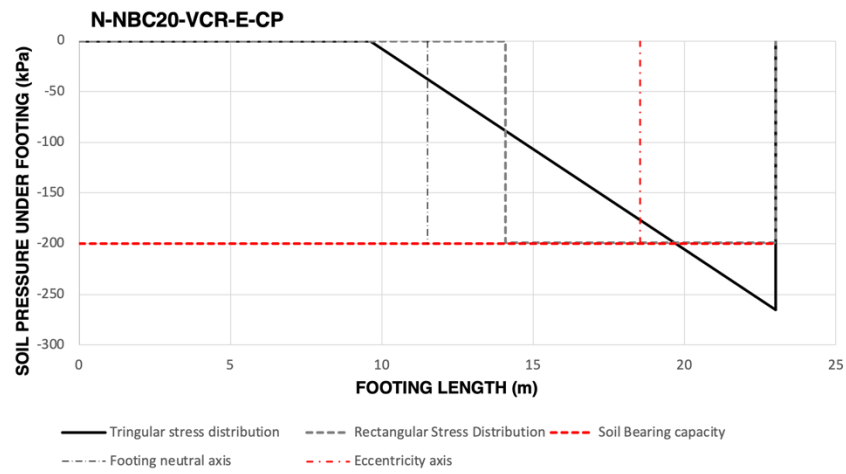


Figure 3.9 Soil's plastic and elastic stress distribution for CP foundation on site Class E.

### 3.6.6 Foundation design summary

In the following table a summary of the foundation dimensions for all cases studied is presented. Note that for all NCP protected foundation, the design moment was the one calculated for  $R_d R_o$  equal to 2.

Table 3.28 Summary of foundation dimensions for studied cases.

| Number of Storeys | Location | Site Class | Foundation type | Design moment | L (m) | B (m) | H (m) |
|-------------------|----------|------------|-----------------|---------------|-------|-------|-------|
| 6                 | MTL      | C          | CP              | $M_{pr}$      | 14.75 | 5     | 1.5   |
| 6                 | MTL      | C          | NCP             | $M_{RdRo=2}$  | 14    | 4.5   | 1.5   |
| 6                 | MTL      | E          | CP              | $M_{pr}$      | 21    | 9     | 1.5   |
| 6                 | MTL      | E          | NCP             | $M_{RdRo=2}$  | 20    | 8.5   | 1.5   |
| 6                 | VCR      | C          | CP              | $M_{pr}$      | 18    | 3.75  | 2     |
| 6                 | VCR      | C          | NCP             | $M_{RdRo=2}$  | 16    | 3.75  | 2     |
| 6                 | VCR      | E          | CP              | $M_{pr}$      | 23    | 10    | 2     |
| 6                 | VCR      | E          | NCP             | $M_{RdRo=2}$  | 22    | 9.5   | 2     |
| 3                 | MTL      | C          | CP              | $M_{pr}$      | 12.5  | 5.5   | 1.5   |
| 3                 | MTL      | C          | NCP             | $M_{RdRo=2}$  | 11.5  | 4.5   | 1.5   |
| 3                 | MTL      | E          | CP              | $M_{pr}$      | 18    | 6     | 1.5   |
| 3                 | MTL      | E          | NCP             | $M_{RdRo=2}$  | 16.5  | 6     | 1.5   |
| 3                 | VCR      | C          | CP              | $M_{pr}$      | 14    | 3.5   | 1.5   |
| 3                 | VCR      | C          | NCP             | $M_{RdRo=2}$  | 13    | 3.4   | 1.5   |
| 3                 | VCR      | E          | CP              | $M_{pr}$      | 17.5  | 6     | 1.5   |
| 3                 | VCR      | E          | NCP             | $M_{RdRo=2}$  | 16    | 6     | 1.5   |

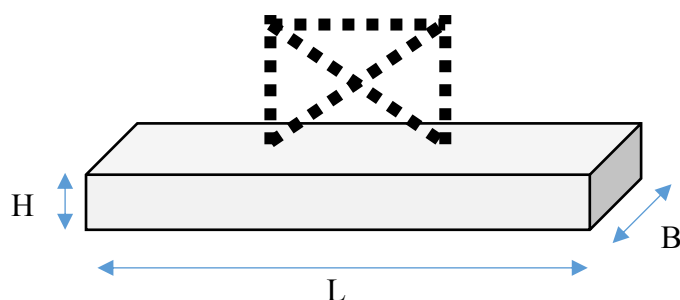


Figure 3.10 Footing's dimensions definition.

### 3.6.7 Foundation rotation and size factor

One of the goals of the study was to investigate the response of the structure considering the minimum required foundation size to meet the soil stress requirements. For comparison purposes, fully compliant with A23.3-19 requirements considering such provision was also designed. The ratio between the volume of fully compliant foundation and the volume of the corresponding foundation that does not meet the increased inter-storey drift limits is calculated and denoted as size factor. A size factor of 1 means that no increase in foundation dimensions is needed to control lateral displacement of the superstructure. Size factor larger than 1 indicates that increased lateral displacement of the superstructure caused by foundation rotation controls foundation dimensions.

Table 3.29 shows the foundation rotation calculated according with A23.3-19.

Table 3.29 Foundation size factor

| Number of Storeys | Location | Site Class | Foundation Type | Foundation 1 rotation | Foundation 2 rotation | Size factor |
|-------------------|----------|------------|-----------------|-----------------------|-----------------------|-------------|
| 3S                | MTL      | C          | CP              | 0.0199                | 0.0171                | 1.05        |
| 3S                | MTL      | C          | NCP             | 0.0212                | 0.0171                | 1.14        |
| 3S                | MTL      | E          | CP              | 0.0199                | 0.0137                | 1.20        |
| 3S                | MTL      | E          | NCP             | 0.0212                | 0.0137                | 1.17        |
| 3S                | VCR      | C          | CP              | 0.0736                | 0.0152                | 1.17        |
| 3S                | VCR      | C          | NCP             | 0.0687                | 0.0152                | 1.20        |
| 3S                | VCR      | E          | CP              | 0.0207                | 0.0128                | 1.20        |
| 3S                | VCR      | E          | NCP             | 0.0205                | 0.0128                | 1.14        |
| 6S                | MTL      | C          | CP              | 0.0307                | 0.0181                | 1.25        |
| 6S                | MTL      | C          | NCP             | 0.0699                | 0.0181                | 1.13        |
| 6S                | MTL      | E          | CP              | 0.0248                | 0.0168                | 1.22        |
| 6S                | MTL      | E          | NCP             | 0.0224                | 0.0168                | 1.24        |
| 6S                | VCR      | C          | CP              | 0.0603                | 0.0164                | 1.37        |
| 6S                | VCR      | C          | NCP             | 0.0724                | 0.0164                | 1.33        |
| 6S                | VCR      | E          | CP              | 0.0254                | 0.0167                | 1.25        |
| 6S                | VCR      | E          | NCP             | 0.0228                | 0.0167                | 1.26        |

### 3.7 Wind verification

Wind design was made to provide a complete and realist condition for the SFRS configuration. Following the NBC 2020 provisions for wind analyses, dynamic procedure was mandatory for some scenarios where the fundamental period was greater than 1 second. Never the less both procedure, static and dynamic were performed for all scenarios. The threshold used to establish if wind demands could impact the design of the building was the factored base shear.

For all cases nominal base shear due to wind effect,  $V_w$ , multiplied by 1.4 as load factor according with NBC load combinations, were smaller than the design base shear due to earthquake effect,  $V_d$ .

As NBC 2020 wind analysis provisions requires, dynamic wind procedures were used when  $T_1$  overpass 1 seconds. MRI of 1/50 years were chosen for ULS calculations an importance factor for wind,  $I_w$ , equal to 1 was considerate. SLS were also verified with 1/10 years MRI, with  $I_w$  equals to 0.75 and a drift limit of 0.20 (h/500).

Table 3.30 Wind analysis results.

| Number of Storeys | Location | Site Class | $T_1$ (s) | Wind analysis procedure | $1.4V_w$ (kN) | $V_d$ (kN) |
|-------------------|----------|------------|-----------|-------------------------|---------------|------------|
| 6S                | MTL      | C          | 1.529     | Dynamic                 | 1410          | 1895       |
| 6S                | MTL      | E          | 1.238     | Dynamic                 | 1378          | 3277       |
| 6S                | VCR      | C          | 0.913     | Static                  | 1329          | 5153       |
| 6S                | VCR      | E          | 0.698     | Static                  | 1329          | 9875       |

Wind effects were evaluated for 6-storey buildings and no further measures were necessary to take because seismic demands govern the design. Because for 3-storey buildings cases wind demands are lower, it was assumed that for those cases, seismic effect will govern the design as well.

## CHAPTER 4 NUMERICAL MODELING

In this chapter details about the construction of the OpenSees-STKO models are presented. Theoretical background, model validation and specific parameters used in the analysis are discussed as well as the implemented resolution procedures. Information regarding ground motion selection and scaling are also provided.

### 4.1 STKO

In this study, Scientific Toolkit for OpenSees (STKO), an advanced GUI, was used to facilitate the preparation of the OpenSees models and for visualize the analysis results. STKO does not run the analyses, instead, it creates the tcl files in which OpenSees models are written. Once the tcl files are created, STKO launches OpenSees to run the analysis. The results are then compressed into a data base that STKO can read and project.

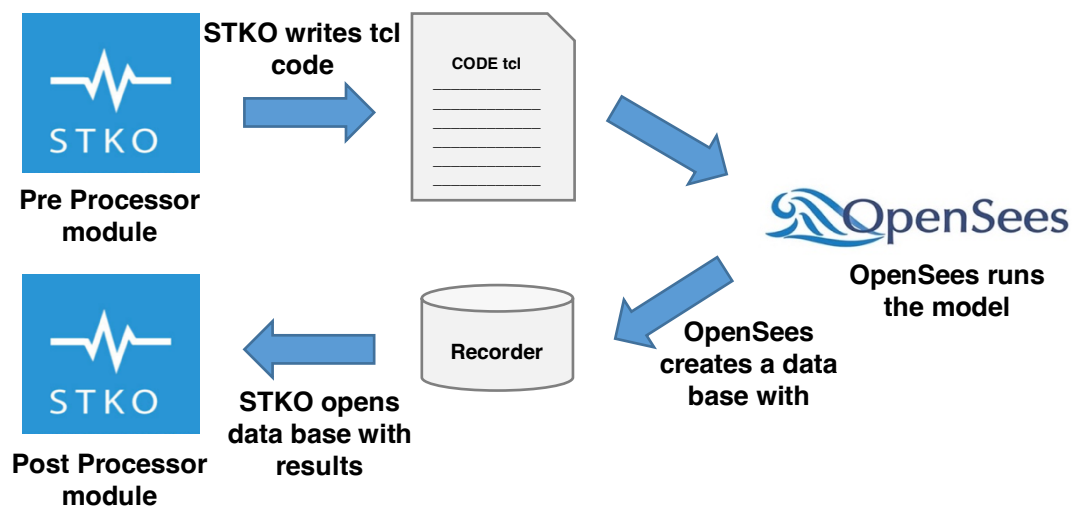


Figure 4.1 STKO implementation diagram

Some advantages beyond the GUI that STKO provide can be summarized as follows: the software is organized according with OpenSees library and tools. It has an extensive library of materials, elements, algorithms, definitions, conditions and physical properties. The uniaxial materials have their virtual testing module for monotonic and cyclic loading. STKO is fully customizable: if a desirable function is needed and it is not built-in inside the STKO environment, the user has the option to write their own tcl script.

## 4.2 Superstructure modeling

CBF structures have been modeled and validated using OpenSees in numerous studies in the past. Uriz and Mahin (2004) and Agüero et al. (2006) proposed a methodology for modelling of CBF inelastic response under cyclic dynamic loading by employing different elements types depending on the expected behaviour and response objectives. Since the main objective of CBF as SFRS is to confine inelastic response into the braces, only these elements are explicitly modeled to exhibit such response. Large-displacements flexural buckling is represented using inelastic fiber cross-section for all diagonals with uniaxial material Giuffré-Meneogotto-Pinto for steel properties. Elastic stiffness and post-yielding parameters are contemplated. As beams and columns are expected to remain elastic, elastic cross-sections have been used for such elements. This allows to reduce the calculation time. Rigid offsets between brace and columns are considered as it has been demonstrated that they influence the response the building (Hsiao 2012). Finally, the effects of gusset-plate connection have been explicitly introduced as well.

### *2D modeling*

Even though Open Sees permits working with 3D models, a single 2D model of the SFRS has been employed. Since the objectives of the study is focused on the response of the SFRS and its foundation, there is no need to develop a complete building modeling.

In-plane stability effects that may occurs have been considered by using a leaning column attached to the SFRS. The leaning column consist of a single elastic truss element for every storey in which the gravitational loading has been applied, including dead, live and snow as specified by NBC 2020. Truss elements have been chosen to avoid any stiffness contribution to the system. Their axial stiffness has been estimated by designing a gravity column of the sample bulling (see Chapter 3) and scaling it by the number of columns required. Finally, leaning columns are attached to the SFRS using truss elements with very high axial stiffness ( $E=2e10$  MPa) to represent a rigid diaphragm action. No continuity moments are developed to avoid any contribution to the lateral strength. Out-of-plane stability is ensured by using lateral supports on the beams and on leaning columns nodes. As lateral bracing for both flanges are placed, this assumption for out-of-plane lateral supports seems adequate.

An additional challenge of the 2D model lies in modelling of torsion. Since design included torsional loads, in the analysis the ground motions have been amplified to account indirectly for

torsional effects. A schematic illustration of the 2D frame is presented in Figure 4.2. Note that subdivision of the elements is not shown for clarity. Central node on diagonals indicate the out-of-straightness maximum location.

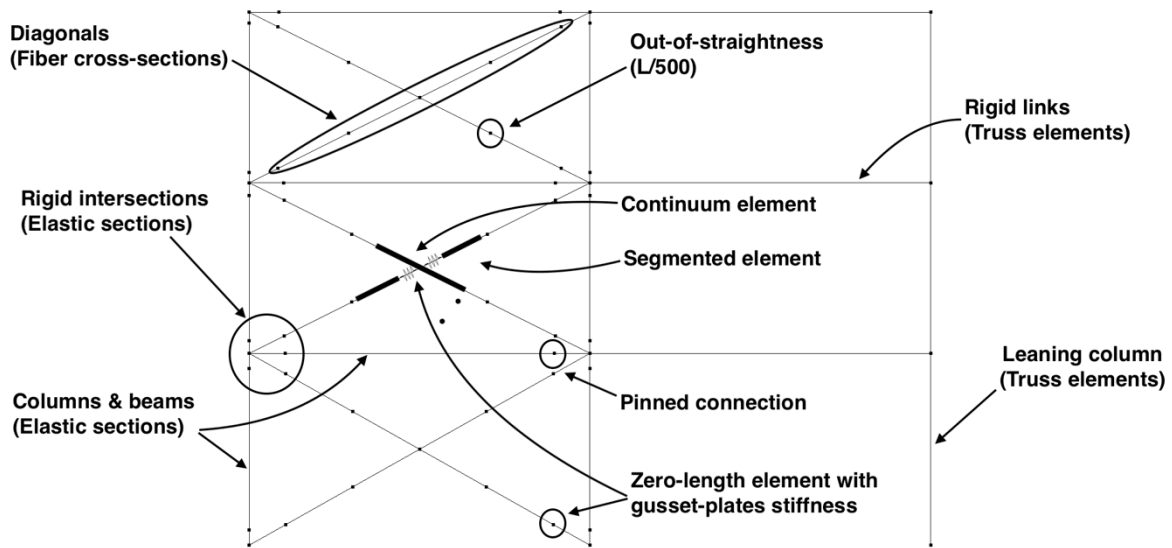


Figure 4.2 OpenSees model of SRFS.

### *Material constitutive law behavior*

As diagonals of CBF are expected to response inelastically, steel constitutive law behavior is defined to reflect the actual response of steel in the post-yielding region of its curve stress-strain illustrated in Figure 4.3.

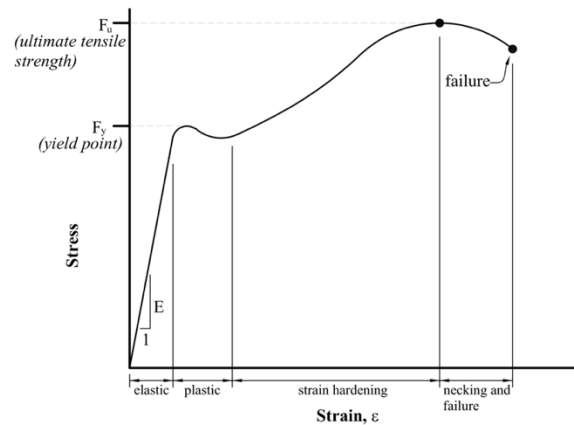


Figure 4.3 Steel stress-strain curve. Adapted from Aghayere (2020).

The Giuffrè-Menegotto-Pinto (GMP) steel constitutive law permits to introduce the inelastic response of steel material in a realistic and accurate way. It has been proved satisfactory for most steel structures under axial and flexure response. Developed by Giuffrè (1970) and improved by Menegotto and Pinto (1973), it considers the nonlinear stress-strain relationships and the Bauschinger effect observed in experimental tests. Elastic modulus  $E$  with a value of 200 000 MPa controls the response for the elastic range. Yielding stress ( $F_y$ ) delimits the elastic and plastic regions. It should be noted that since capacity design has been employed, in the analysis,  $F_y$  is set as the expected probable yielding stress of the braces according to CSA S16-19, i.e.  $F_y = 460$  MPa for HSS.

Contrary to a bi-linear model, GMP allows a more realistic transition between elastic and plastic regions by using the parameters  $R_0$ ,  $cR_1$  and  $cR_2$ .  $R_0$  whose values varies between 15 to 30. Agüero et al. (2004) demonstrated that taking  $R_0$  equal to 20 represents well the post-buckling behavior of CBF. Fixed values for  $cR_1$  and  $cR_2$  (0.925 and 0.15, respectively) are recommended. Kinematic strain-hardening is introduced as a percentage of  $E$ . As for an isotropic hardening, experimental study by Agüero et al. (2004) showed that it does not have a significant effect on the response. For this reason, isotropic strain hardening was not considered in this study. Combined

effect of flexure and axial force can be well represented using Steel02, however shear interaction cannot be modelled. Because the inelastic mechanism of CBF involve yielding in tension and flexural buckling in compression, this constitutive law is considered adequate for the purposes for this study.

### *Fiber cross section*

To model flexural buckling of the braces, the overall cross section of the braces is discretized into a series of fibers which will behave individually according to the material constitutive law. The response of the complete cross section can be estimated as the sum of the response of each fiber in every integration point of the element (Spacone et al. 1996). It should be noted that fibers cross section does not permit to explicitly consider the torsion resistance of the elements itself. OpenSees tool *Aggregator* permits to manually add the torsional stiffness of the elements as the product of its shear modulus ( $G$ ) and torsional constant ( $J$ ).  $G$  is taken as 77 000 MPa for steel and  $J$  can be taken from the Canadian Steel Construction Handbook or calculated for each section by material mechanics procedures.

Fiber cross-sections permit to implement distributed plasticity. In other words, with such discretization, an inelastic response can be determined along the element of interest, contrary to the case when concentrated plasticity modeling is used which only permits a plasticisation in a single point. However, this distributed plasticity is only spread out into defined series of points along the length of the element known as integration point in which equilibrium is reached as Figure 4.4 (d) portrays. As Uriz and Mahin (2004) and Agüero et al. (2006) suggest, a discretization in 16 fiber is accurate enough to represent well the response of braces under seismic loads.

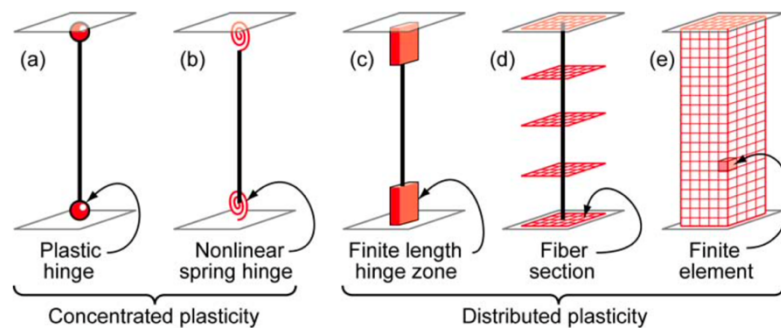


Figure 4.4 Idealized distribution of plasticity. Adapted from NEHRP (2010).

It should be noted that in order to trigger the out-of-plane buckling of the diagonals, an out-of-straightness must be defined for the diagonals. As per CSA S16-19, a geometric imperfection equal to  $L/500$ , where  $L$  is the length of the considered element, seems adequate to trigger flexural buckling and compute inelastic response with accuracy.

*Elastic elements and rigid segments.*

As discussed previously, beam and columns are represented using elastic elements with the cross-section dimensions obtained by design. Only elasticity modulus  $E=200000$  MPa is needed to characterize their response. For the rigid segments at the intersection of the beam or brace with columns, the same cross-section of the element was used (column, beam, or diagonal) but using  $E=2e10$  MPa in order to achieve a rigid response of these elements. To define the length of the rigid segments, the design of connections was carried out to obtain realistic dimension of the connecting plates. Next, the criteria suggested by Hsiao (2012) was applied. The horizontal length corresponding to the beam segment is set as 0.75 times the distance from the column axis to the tipping point of the gusset-plate ( $0.75 \cdot A$ ). The vertical length equals the height of the gusset-plate plus half beam depth ( $B$ ), while diagonal segments span from the work point of the intersection to the edge of the HSS ( $C$ ).

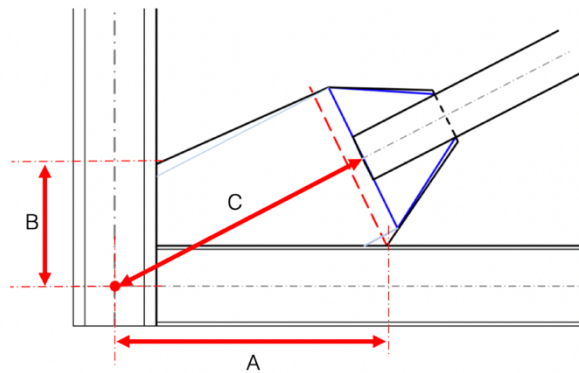


Figure 4.5 Definition of the rigid segments at beam-column intersections.

### Beam-column elements

OpenSees offers two formulations to solve beam-columns integration algorithms: (i) displacement-based element formulation (DBE) and (ii) force-based element formulation (FBE).

DBE approach involves interpolating section deformations based on an approximate displacement field by the finite element method and using the Principle of Virtual Displacements (PVD) to establish the element equilibrium relationship. To approximate the nonlinear response of the element, it is necessary to enforce a constant axial deformation and a linear curvature distribution along the element length. It is important to note that this method is exact only for prismatic linear elastic elements. For more complex elements, mesh refinement is necessary to accurately represent higher order distributions of deformations.

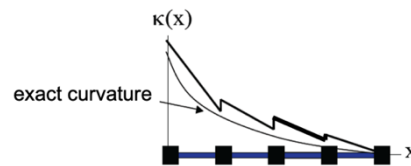


Figure 4.6 DBE solution compared with the exact solution . Adapted from Terzic (2011).

FBE approach involves the use of the exact equilibrium solution within the basic system of a beam-column element. This approach ensures that equilibrium is maintained between element and section forces, and it remains accurate within the range of constitutive nonlinearity. The determination of section forces is achieved through interpolation within the basic system, which is based on static equilibrium. In the absence of distributed element loads, the interpolation yields a constant axial force and a linear distribution of bending moment. To ensure compatibility between section and element deformations, the Principle of Virtual Forces (PVF) is used.

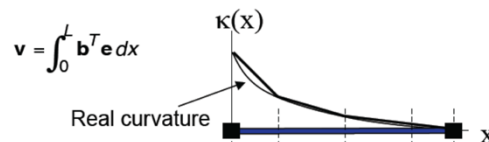


Figure 4.7 FBE solution compared with the exact solution. Adapted from Terzic (2011).

Neuenhofer and Filippou (1997) performed a parametric study to evaluate the accuracy of each approach and found that for FBE the accuracy of a solution can be improved through two methods: increasing the number of integration points or increasing the number of elements. In fact, it has been shown that with only one element and seven integration points, the FBE method yields an error of less than 2% for both global and local response quantities. From computational perspective, increasing the number of integration points is preferred than increasing the number of elements.

When DFE approach is used, the accuracy of the solution can only be improved by increasing the number of elements. This is attributed to the fact that DBE uses displacement interpolation functions that approximate the exact solution and involve both discretization and numerical error, while FBE method utilizes exact force interpolation functions, thereby minimizing discretization errors and limiting numerical errors.

Consequently, FBE are most commonly used since they give a more reliable estimation of response and with smaller computational efforts.

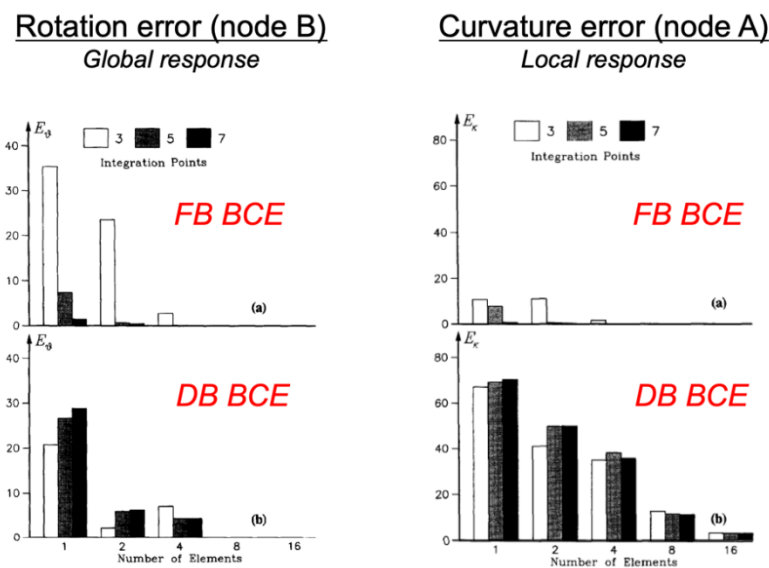


Figure 4.8 Results of parametric study on FBE and DBE by Neuenhofer and Filippou (1997). Adapted from Terzic (2011).

### *Experimental validation*

Agüero et al. (2006) calibrated parameters of the OpenSees model for CBF against the values obtained by experimental tests to achieve reliable CBF response under inelastic cyclic demands. A single-storey building with X-bracing was subjected to an earthquake ground motion and the response parameters were compared with OpenSees results. The recommendation from this study regarding the number of fibres (16), number of integration points (8), and number of elements (8) were used in the CBF model constructed for the present study. Same was done to specify Steel02 user-defined parameter as explained previously in Section 0. Hierarchy of the modeling procedure is illustrated in Figure 4.9.

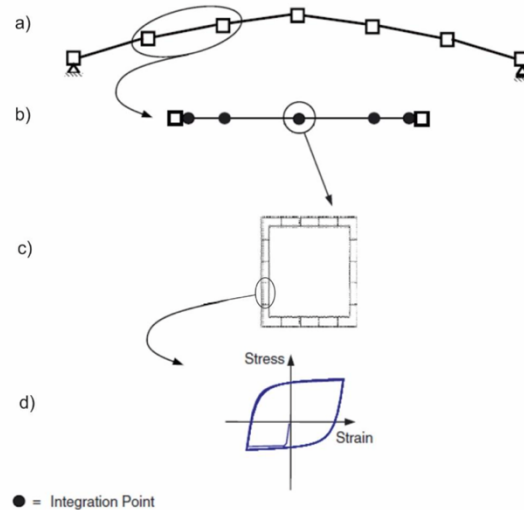


Figure 4.9 Modeling of inelastic response of braces to achieve distributed plasticity approach. Adapted from Uriz and Mahin (2008)

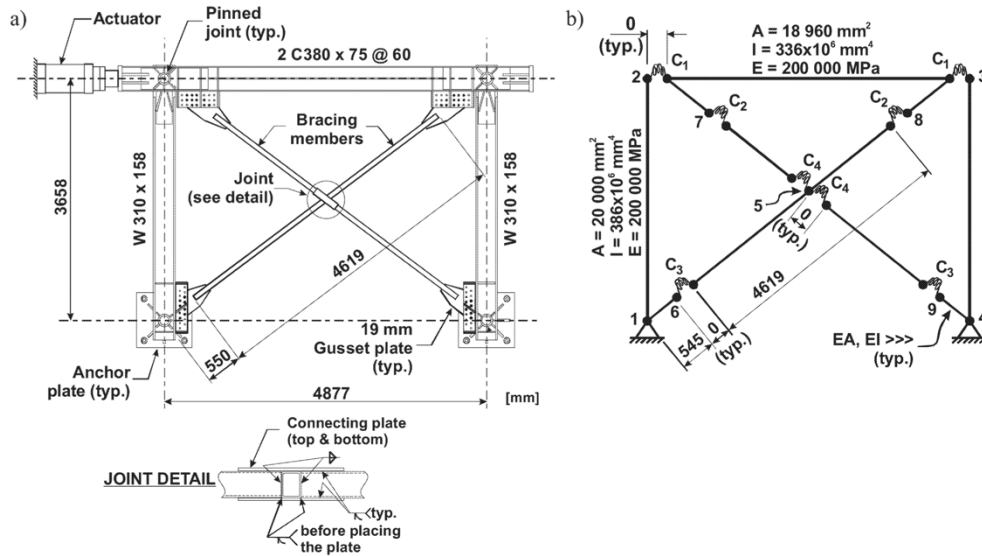


Figure 4.10 Experimental and numerical model of parametric assessment. Adapted from Agüero et al. (2006).

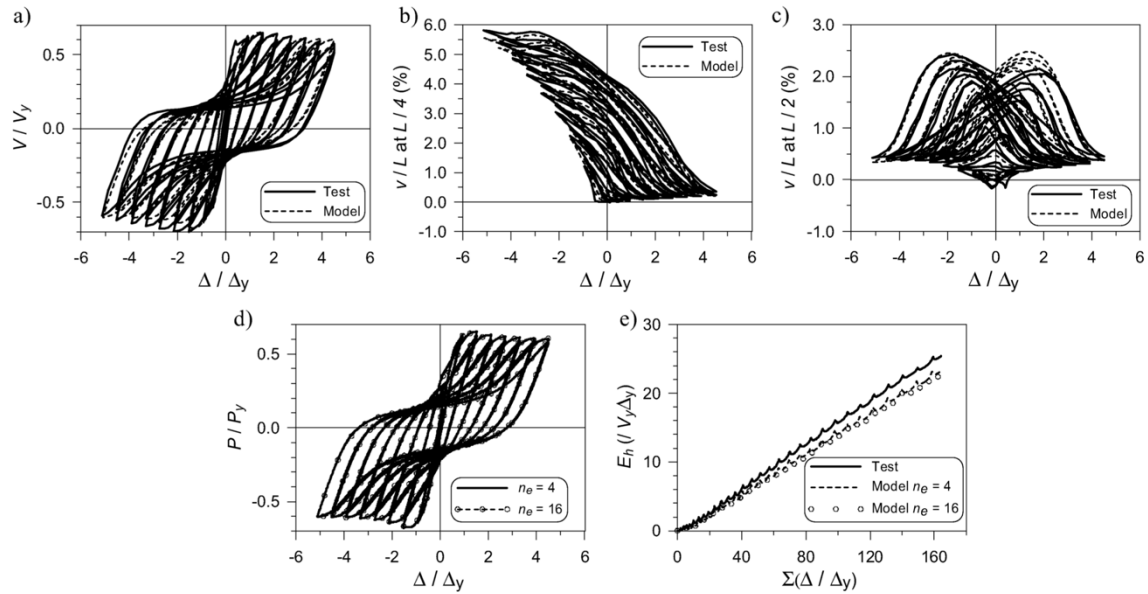


Figure 4.11 Experimental and numerical results of parametric assessment. Adapted from Agüero et al. (2006).

The importance on the structure parameters lies principally on the inelastic response since a certain amount of ductility demand is expected due to the yielding and/or the buckling of braces. Even if the main effort of this study is focused on the foundations and soil interaction, a proper structural response is needed as well in order to accurately measure the influence of SSI on the structure.

The accuracy of the modelling methodology has been validated by Agüero et Al. (2006) through a parametric study which involves laboratory testing braces and several OpenSees simulations seeking to match results with the tested probes. They have shown that 4 sub-elements at each diagonal segment allowed to buckle, i.e. 8 sub-elements for the entire diagonal, provides enough accurate to reflect the actual behavior of the diagonal while keeping computational efforts at a reasonable level. Following the modeling hierarchy, 4 integrations points, or Gauss point, has been shown as the minimum needed to provide a convergence of the results. Equilibrium equations are solved at each integration considering the cross-section properties of the diagonal, for this matter, the refinement of the fiber cross-section play a significant role as well. They observed a maintained response trend from finer fiber cross-section meshing down to 16 fibers. Lesser fiber number resulted in the errors greater than 1%. As a results 16 fibers are seeming as reasonable value for the scope of this study. Finally, in order to trigger out-of-plane buckling, an out-of-straightness with magnitude of 1/500 was specified as well.

Table 4.1 Agüero et Al. (2006) modeling parameters.

|                     |       |       |
|---------------------|-------|-------|
| Sub-elements        | $n_e$ | 8     |
| Integration points  | $n_i$ | 4     |
| Fiber elements      | $n_f$ | 16    |
| Out-of-straightness | --    | 1/500 |

The last parameter linked to the hierarchy chain for diagonals modeling, are those related to GMP material. Are introduced in the literature review chapter, Steel02 was used to characterize the mechanical behavior of brace's material. Test results performed by Agüero et Al. (2006) indicated the user defined parameter needed to describe the inelastic response of material. They conclude dismiss that isotropic strain hardening seems appropriate since it was not observed in laboratory testing. R0 parameter related to Bauschinger effect gave better relts when it has dropped from 25 to 20. Kinematic strain hardeing of 1% percent (0.01) was allso obervsed. By the other hand, code

defined parameter related to the strength,  $F_y$ , and elastic range stiffness,  $E_0$ , shall meet code requirements.  $F_y$  equals to 460 is set by CSA S16 as lower bound for HSS  $R_y F_y$  probable yielding stress while 200000 MPa for the elastic modulus is establish as standard for structural steel.

Table 4.2 Steel02 user and code defined parameters

| Parameter type | Parameter | Value  | Unit |
|----------------|-----------|--------|------|
| Code defined   | $F_y$     | 460    | MPa  |
| Code defined   | $E_0$     | 200000 | MPa  |
| User defined   | $R_0$     | 20     | --   |
| User defined   | CR1       | 0.925  | --   |
| User defined   | CR2       | 0.15   | --   |
| User defined   | b         | 0.01   | --   |

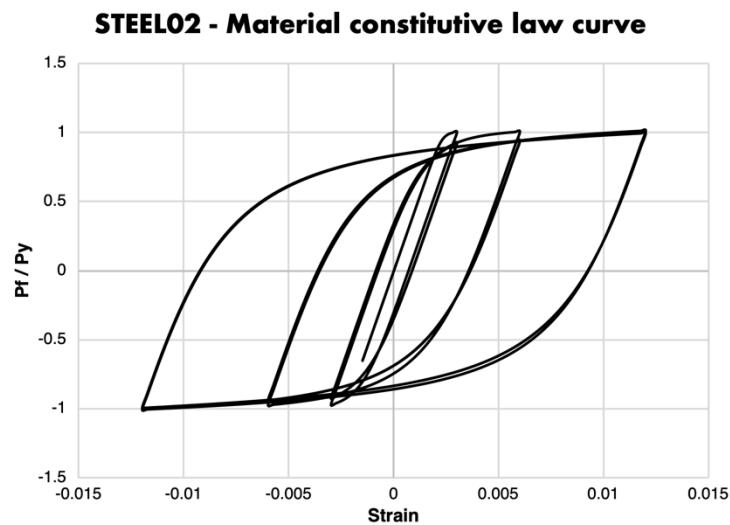


Figure 4.12 Steel02 normalized hysteretic curve used for modeling.

NEHRP guidelines for non-linear modeling recommend to use Raleigh proportional damping which is based in the assumption that the damping ratio for two modes are known. The method allows to build a damping matrix, (see equation 4.1), as the sum of a proportional damping matrix related to the stiffness and another matrix related to the proportional modal mass mobilized.

$$[C] = a_0[M] + a_1[K] \quad (4.1)$$

$a_0$  and  $a_1$  are proportional parameters related to mass and stiffness respectively. They can be estimated by knowing specific critical damping portion,  $\xi$  at two modes, generally taken the first two modes, since they contribute substantially more to the complete response. NEHRP guidelines recommends a range of  $\xi$  between 1% up to 5% depending of the building characteristics. Most commonly  $\xi$  is taken as 5%, but for steel buildings responding in inelastic range is recommended in the literature to take  $\xi$  equal to 3%.  $a_0$  and  $a_1$  can be estimated from equations 4.2 and 4.3 where the subscript  $i$  denotes the number of the first mode considered, the subscript,  $j$ , the number of the consecutive mode considered, and  $\omega$  denotes the modal angular frequency. For every modal angular frequency, the damping ratio,  $\xi$ , can be calculated using equation 4.4.

$$a_0 = \xi \frac{2\omega_1\omega_j}{\omega_1 + \omega_j} \quad (4.2)$$

$$a_1 = \xi \frac{2}{\omega_1 + \omega_j} \quad (4.3)$$

$$\xi_i = \frac{a_0}{2\omega_i} + \frac{a_1\omega_i}{2} \quad (4.4)$$

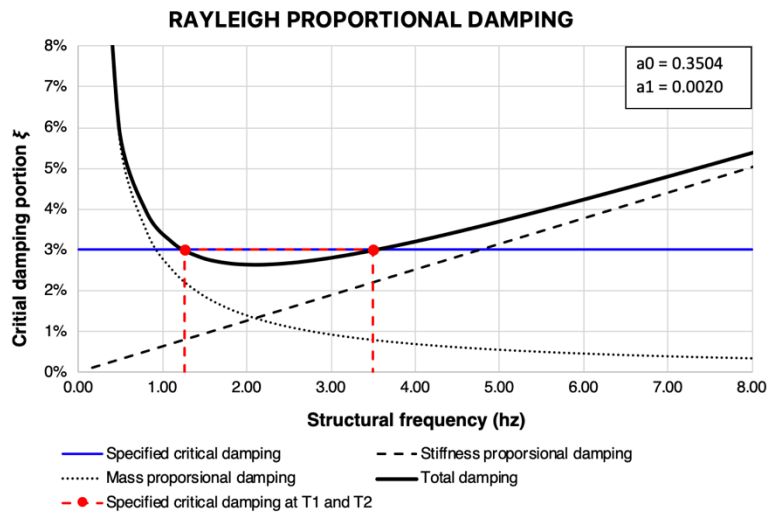


Figure 4.13 Damping ratio  $\xi$  as a function of frequency for Rayleigh-type damping.

### *Zero-length elements*

In the OpenSees element library there is a special purpose element with no length whose purpose is to model the “connection” between two finite length elements. These zero-length elements are used to represent an interaction following a user defined behaviour by either a uniaxial material in a desirable degree of freedom or by an ensemble of uniaxial materials for different degrees of freedom.

### *Connections*

For design purposes, diagonals are considered to be pinned at both ends. In reality the brace connection provides certain fixity that can have an impact on structural response. For this reason, a special attention is devoted to connection modelling. One approach to modelling consist of adding and inelastic spring with GMP Steel02 material as link between the diagonal fiber cross-section and the diagonal elastic rigid segment. Hsiao (2012) validated this approach by experimental testing. It has been shown that the elastic response of the gusset-plate due to bending out-of-plane of the frame can be estimated by classical stress bending distribution up to fully plastification of the gusset-plate steel. From that point, a kinematic hardening of about 1% of the elastic modulus slope have been observed.

The yielding moment in which the gusset-plate has an inelastic response can be expressed as;

$$M_y = F_{y,gusset} \left( \frac{W_w t^2}{6} \right) \quad (4.5)$$

where:

$F_{y,gusset}$  = yielding stress of the gusset-plate steel.

$W_w$  = Whitmore section width of the gusset plate with a 45° projection.

$t$  = thickness of the gusset-plate.

$M_y$  = moment needed to reach  $F_y$ .

$M_y$  is used to define the strength related parameter of the material constitutive law. The stiffness related parameter,  $K_{flexure}$ , it can be calculated using the following equation:

$$K_{flexure} = \frac{EI}{L} \quad (4.6)$$

where:

$K_{\text{flexure}}$  = rigidity by flexure of a give element.

$E$  = elastic modulus of the material element.

$I$  = moment of inertia of the cross section of the element about the bending axis.

$L$  = length of the element.

By expressing the moment of inertia,  $I$ , in function of the dimensions of the gusset plate, the flexural stiffness of the gusset plate  $K_{\text{rotational}}$  is given by:

$$K_{\text{rotational}} = \frac{E}{L_{\text{ave}}} \left( \frac{W_w t^3}{12} \right) \quad (4.7)$$

where:

$K_{\text{rotational}}$  = rigidity by flexure of the gusset-plate.

$L_{\text{ave}}$  = Average of the length  $L_1$ ,  $L_2$  and  $L_3$  shown in figure Figure 4.14.

These two parameters define properly the anticipated response of the connection up to yielding. For the inelastic response strain hardening should be specified. A kinematic hardening of 1% suggested in literature (Hsiao, 2012) was adopted in this study. In order to implement these parameters into the OpenSees model, GMP Steel02 uniaxial material has been used with the following material parameters:

Table 4.3 Gusset-plate modelling parameter

| Steel02<br>parameter | Gusset-plate<br>parameter |
|----------------------|---------------------------|
| $F_y$                | $M_y$                     |
| $E$                  | $K_{\text{rotational}}$   |
| $b0$                 | 0.01                      |
| $R0$                 | 20                        |
| $cR1$                | 0.925                     |
| $cR2$                | 0.15                      |

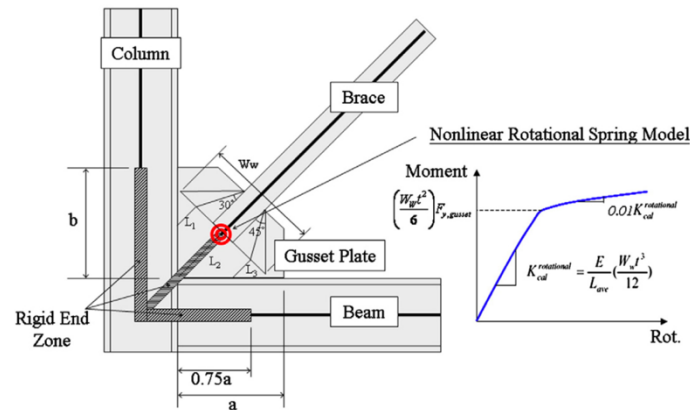


Figure 4.14 Gusset-plate connection as inelastic spring using steel02. Adapted from Hsiao (2012).

Tables from Table 4.4 up to Table 4.11 show the vertical, horizontal and diagonal rigid extension tagged as *Link V*, *Link H* and *Link D* respectively. Also, they present the flexural yielding moment for the gusset plate,  $M_y$ , and its rotational stiffness out-of-plane,  $K_{ROT}$ .

Table 4.4 Rigid extensions and ZLE properties for scenario 6S-NBC20-MTL-C

| Storey | Lower  |        |        |          |           | Upper  |        |        |          |           |
|--------|--------|--------|--------|----------|-----------|--------|--------|--------|----------|-----------|
|        | Link V | Link H | Link D | $M_y$    | $K_{ROT}$ | Link V | Link H | Link D | $M_y$    | $K_{ROT}$ |
|        | mm     | mm     | mm     | N-mm     | N-mm      | mm     | mm     | mm     | N-mm     | N-mm      |
| 6      | 226    | 683    | 775    | 7081591  | 121711198 | 226    | 686    | 778    | 7081591  | 121345191 |
| 5      | 276    | 760    | 872    | 11692062 | 200465951 | 276    | 760    | 872    | 11692062 | 200465951 |
| 4      | 305    | 889    | 967    | 11692062 | 198024061 | 305    | 856    | 918    | 11692062 | 205736676 |
| 3      | 309    | 896    | 982    | 15271265 | 286299162 | 309    | 896    | 982    | 15271265 | 286299162 |
| 2      | 314    | 906    | 983    | 15271265 | 289380372 | 314    | 905    | 980    | 15271265 | 289863920 |
| 1      | 367    | 721    | 683    | 18325518 | 418795731 | 376    | 999    | 1106   | 28633622 | 571303528 |

Table 4.5 Rigid extensions and ZLE properties for scenario 6S-NBC20-MTL-E

| Storey | Lower  |        |        |          |           | Upper  |        |        |          |            |
|--------|--------|--------|--------|----------|-----------|--------|--------|--------|----------|------------|
|        | Link V | Link H | Link D | $M_y$    | $K_{ROT}$ | Link V | Link H | Link D | $M_y$    | $K_{ROT}$  |
|        | mm     | mm     | mm     | N-mm     | N-mm      | mm     | mm     | mm     | N-mm     | N-mm       |
| 6      | 226    | 715    | 823    | 7081591  | 116644703 | 226    | 686    | 779    | 7081591  | 121199648  |
| 5      | 290    | 812    | 965    | 23861352 | 513850347 | 290    | 811    | 962    | 23861352 | 514629268  |
| 4      | 372    | 999    | 1107   | 28342619 | 565775050 | 372    | 999    | 1107   | 28342619 | 565775050  |
| 3      | 372    | 999    | 1107   | 28342619 | 565775050 | 372    | 999    | 1107   | 28342619 | 565775050  |
| 2      | 406    | 1045   | 1167   | 37463587 | 763579965 | 377    | 995    | 1106   | 34294569 | 742252369  |
| 1      | 469    | 868    | 870    | 37014152 | 795405369 | 445    | 1104   | 1244   | 49109942 | 1003140796 |

Table 4.6 Rigid extensions and ZLE properties for scenario 6S-NBC20-VCR-C

| Storey | Lower  |        |        |          |           | Upper  |        |        |          |           |
|--------|--------|--------|--------|----------|-----------|--------|--------|--------|----------|-----------|
|        | Link V | Link H | Link D | $M_y$    | $K_{ROT}$ | Link V | Link H | Link D | $M_y$    | $K_{ROT}$ |
|        | mm     | mm     | mm     | N-mm     | N-mm      | mm     | mm     | mm     | N-mm     | N-mm      |
| 6      | 226    | 683    | 775    | 7081591  | 121711198 | 226    | 686    | 778    | 7081591  | 121345191 |
| 5      | 269    | 782    | 909    | 11378404 | 191125644 | 274    | 756    | 874    | 14861588 | 286706947 |
| 4      | 309    | 895    | 982    | 15271265 | 286181888 | 309    | 893    | 979    | 15271265 | 286654798 |
| 3      | 318    | 908    | 1010   | 23861352 | 525893682 | 318    | 908    | 1010   | 23861352 | 525893682 |
| 2      | 386    | 1011   | 1119   | 29622859 | 581105771 | 386    | 1011   | 1119   | 29622859 | 581105771 |
| 1      | 383    | 741    | 719    | 29331856 | 767274326 | 383    | 1006   | 1114   | 29331856 | 579011706 |

Table 4.7 Rigid extensions and ZLE properties for scenario 6S-NBC20-VCR-E

| Storey | Lower  |        |        |          |            | Upper  |        |        |           |            |
|--------|--------|--------|--------|----------|------------|--------|--------|--------|-----------|------------|
|        | Link V | Link H | Link D | $M_y$    | $K_{ROT}$  | Link V | Link H | Link D | $M_y$     | $K_{ROT}$  |
|        | mm     | mm     | mm     | N-mm     | N-mm       | mm     | mm     | mm     | N-mm      | N-mm       |
| 6      | 226    | 715    | 823    | 7081591  | 116709341  | 226    | 685    | 777    | 7081591   | 121511529  |
| 5      | 290    | 817    | 972    | 23861352 | 511757727  | 290    | 811    | 963    | 23861352  | 514861672  |
| 4      | 499    | 1235   | 1415   | 56149381 | 1012883788 | 499    | 1191   | 1350   | 56149381  | 1044966713 |
| 3      | 499    | 1241   | 1424   | 56149381 | 1008612509 | 499    | 1235   | 1415   | 56149381  | 1012883788 |
| 2      | 540    | 1290   | 1491   | 82129197 | 1599445370 | 540    | 1290   | 1491   | 82129197  | 1599445370 |
| 1      | 532    | 956    | 986    | 60339818 | 1336626789 | 549    | 1303   | 1520   | 107270788 | 2301604147 |

Table 4.8 Rigid extensions and ZLE properties for scenario 3S-NBC20-MTL-C

| Storey | Lower  |        |        |          |           | Upper  |        |        |          |           |
|--------|--------|--------|--------|----------|-----------|--------|--------|--------|----------|-----------|
|        | Link V | Link H | Link D | $M_y$    | $K_{ROT}$ | Link V | Link H | Link D | $M_y$    | $K_{ROT}$ |
|        | mm     | mm     | mm     | N-mm     | N-mm      | mm     | mm     | mm     | N-mm     | N-mm      |
| 3      | 249    | 702    | 756    | 7081591  | 128876909 | 249    | 702    | 756    | 7081591  | 128876909 |
| 2      | 297    | 813    | 912    | 14861588 | 287667656 | 297    | 813    | 912    | 14861588 | 287667656 |
| 1      | 352    | 604    | 593    | 26771377 | 851354986 | 352    | 840    | 953    | 26771377 | 625777434 |

Table 4.9 Rigid extensions and ZLE properties for scenario 3S-NBC20-MTL-E

| Storey | Lower  |        |        |          |            | Upper  |        |        |          |           |
|--------|--------|--------|--------|----------|------------|--------|--------|--------|----------|-----------|
|        | Link V | Link H | Link D | $M_y$    | $K_{ROT}$  | Link V | Link H | Link D | $M_y$    | $K_{ROT}$ |
|        | mm     | mm     | mm     | N-mm     | N-mm       | mm     | mm     | mm     | N-mm     | N-mm      |
| 3      | 254    | 707    | 766    | 9638832  | 197789792  | 254    | 707    | 766    | 9638832  | 197789792 |
| 2      | 313    | 867    | 997    | 23861352 | 518404560  | 313    | 867    | 997    | 23861352 | 518404560 |
| 1      | 372    | 628    | 624    | 33942456 | 1108593827 | 372    | 863    | 984    | 33942456 | 829487402 |

Table 4.10 Rigid extensions and ZLE properties for scenario 3S-NBC20-VCR-C

| Storey | Lower  |        |        |          |            | Upper  |        |        |          |           |
|--------|--------|--------|--------|----------|------------|--------|--------|--------|----------|-----------|
|        | Link V | Link H | Link D | $M_y$    | $K_{ROT}$  | Link V | Link H | Link D | $M_y$    | $K_{ROT}$ |
|        | mm     | mm     | mm     | N-mm     | N-mm       | mm     | mm     | mm     | N-mm     | N-mm      |
| 3      | 249    | 703    | 757    | 7081591  | 128618888  | 249    | 703    | 757    | 7081591  | 128618888 |
| 2      | 313    | 835    | 951    | 23861352 | 534124671  | 313    | 835    | 951    | 23861352 | 534124671 |
| 1      | 387    | 653    | 655    | 35703022 | 1112897306 | 387    | 857    | 968    | 35703022 | 869866348 |

Table 4.11 Rigid extensions and ZLE properties for scenario 3S-NBC20-VCR-E

| Storey | Lower  |        |        |          |            | Upper  |        |        |          |           |
|--------|--------|--------|--------|----------|------------|--------|--------|--------|----------|-----------|
|        | Link V | Link H | Link D | $M_y$    | $K_{ROT}$  | Link V | Link H | Link D | $M_y$    | $K_{ROT}$ |
|        | mm     | mm     | mm     | N-mm     | N-mm       | mm     | mm     | mm     | N-mm     | N-mm      |
| 3      | 253    | 741    | 765    | 7081591  | 132308222  | 253    | 741    | 765    | 7081591  | 132308222 |
| 2      | 318    | 873    | 959    | 23861352 | 545363324  | 318    | 873    | 959    | 23861352 | 545363324 |
| 1      | 376    | 669    | 637    | 33942456 | 1132950450 | 376    | 904    | 997    | 33942456 | 842924398 |

### 4.3 Soil-structure interaction modeling

It is a common practice for building analysis to impose fully restrained boundary conditions at the bases of columns. In order to implement the SSI into the system, the structure is attached to its foundation instead. The foundation consists of an elastic beam-column element with length equal to the length of the footing. The transversal cross-section of this element corresponds to the out-of-plane dimension of the footing. Since steel reinforcement is provided to sustain internal stresses of the footing, elastic response is expected (Moehle, 2015). In consequence, elastic elements seem reasonable to characterize the footing response.

The foundation beam-column element is discretized into a set of internal nodes. At each node a qz spring is placed. These springs are attached to the foundation element from a fully restrained node. By using uniaxial material Qzsimple2 as constitutive material of the springs in the vertical translational DOF, the soil response can be estimated through the spring's responses. The springs' material parameters, as well as their distribution are selected to match the vertical and rotational stiffness of the soil. OpenSees provides the environment needed to perform such modeling with Zero-length elements (ZLE); a type of element that can respond in any desirable DOF following the specified material constitutive law. It should be noted that it is possible to place horizontal springs as well to account for the sliding resistance of the soil. However, in the present study, sliding was not modeled as the previous studies (Murugananthan, 2018) have showed that this was not a governing mode for foundation behaviour.

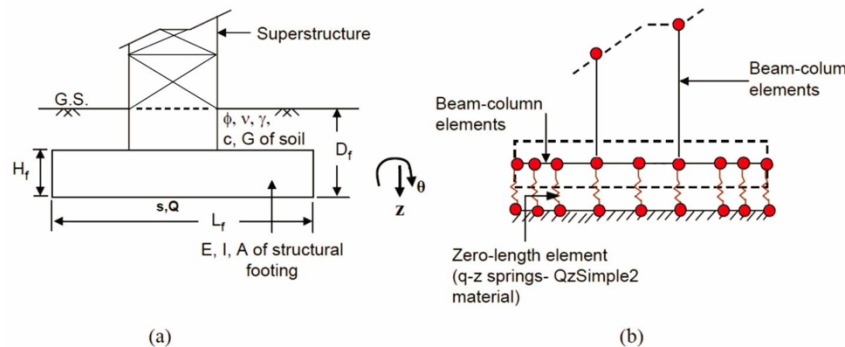


Figure 4.15 System Soil-Column-Foundation. a) Schematic representation of the system b) OpenSees modeling of the system. Adapted from Raychowdhury et Hutchinson (2008)

*Spring material constitutional law behavior*

OpenSees has an extensive library of materials for several application. among others QzSimple1 uniaxial material developed by Boulanger (1999). This material was developed to reproduce the response of foundation piles for cases for which the compressive behaviour of the soil differs significantly from the uplift behaviour. QzSimple1 material has an asymmetric hysteretic backbone curve defined by the compressive resistance of the soil and a reduced capacity under tension loading. It consists of an elastic spring, a radial damper parallel to the elastic spring, a plastic spring with the role to capture the permanent inelastic displacement, and finally, a closure and a drag springs intended to capture the tension resistance of the soil if it occurs. Components of QzSimple1 material are illustrated in Figure 4.16. Since coordinates and directions in OpenSees environment are arbitrary, the local orientation of the qz springs is defined by the initial and final node of the ZLE. Commonly the fixed node is the initial node and foundation node is set as the de final one.

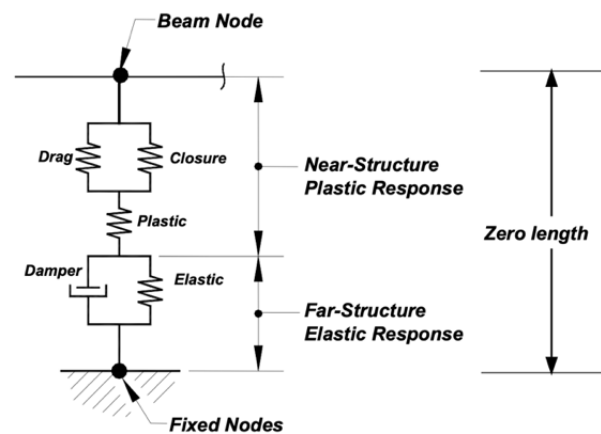


Figure 4.16 QzSimple1 components. Adapted from Boulanger (2000).

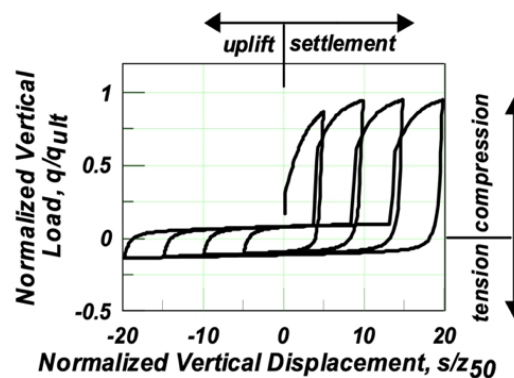


Figure 4.17 Cyclic axial displacement response of zero-length spring models using QzSimple1 material. Adapted from Boulanger (2000).

Figure 4.18 shows two principal regions of the QzSimple backbone curve: (i) the elastic response and (ii) the plastic response. The elastic region is characterized by equation 4.8 while plastic region is determinate with equation 4.9.

$$q = k_{in}z \quad (4.8)$$

$$q = q_{ult} - (q_{ult} - q_0) \left[ \frac{c * z_{50}}{c * z_{50} + |z - z_0|} \right]^n \quad (4.9)$$

$$q_0 = C_r q_{ult} \quad (4.10)$$

$$z_{50} = \frac{k_{fac}}{k_{in}} * q_{ult} \quad (4.11)$$

where:

$k_{in}$  = initial stiffness of the soil estimated by the impedance equation from Gazetas and Stokoe (1991).

$k_{fac}$  = factored stiffness of the soil, set as 1.21

$z$  = vertical displacement of the spring

$q$  = resultant force due to displacement

$q_{ult}$  = soil ultimate bearing resistance

$q_0$  = soil yielding resistance

$z_{50}$  = displacement at which 50% of the ultimate load is mobilized

$C_r, c, n$  = constitutive parameters controlling the shape of the post-yield portion of the backbone curve

$z_0$  = displacement at yielding point

These equations are applicable to both uniaxial materials, QzSimple1 and QzSimple2, the only difference being the constitutive parameters  $C_r$ ,  $c$  and  $n$ . These parameters have been determined for QzSimple2 by Raychowdhury et Hutchinson (2008) through a regression analysis of an ensemble of shallow foundation laboratory tests. For sand type soils,  $c$  has been determined as 9.29,  $n$  as 5.5 and  $C_r$  as 0.36.

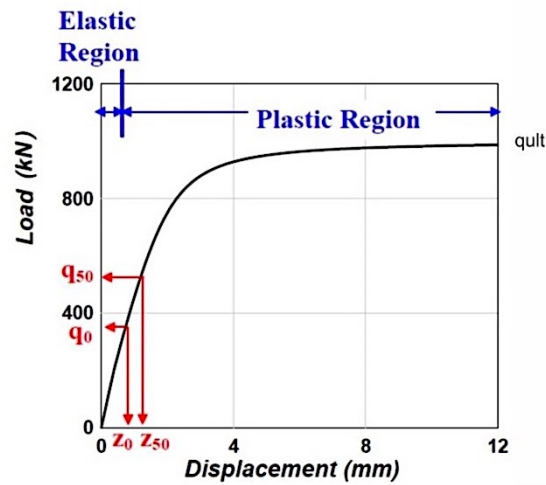


Figure 4.18 Inelastic backbone curve of QzSimple1. Adapted from Raychowdhury et Hutchinson (2008).

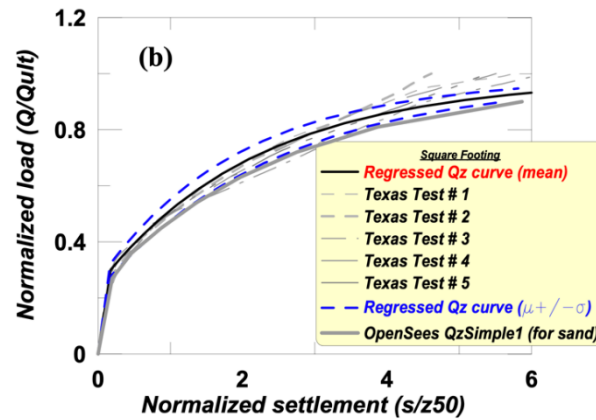


Figure 4.19 Regression analysis result of an ensemble of tests of shallow foundations. Adapted from Raychowdhury et Hutchinson (2008).

Since an asymmetrical hysteric cyclic response of soil is expected due to its small resistance under tensile loading, closure and drags springs are placed as well. The drag component ( $q^d$ ) provides the tensile resistance while the closure component ( $q^c$ ) is a bilinear elastic spring, which is relatively rigid in compression and extremely flexible in tension. An user defined parameter ( $C_d$ ) allows the user to specify the tensile resistance as portion of compressive bearing resistance, permitting, if desirable, to neglect the tensile resistance completely. In the present study, the tensile resistance of the soil has been neglect, therefore  $C_d$  ratio has been taken as 0.

$$q^d = C_d * q_{ult} - (C_d * q_{ult} - q_0) \left[ \frac{z_{50}}{z_{50} + 2 * |z - z_0|} \right] \quad (4.12)$$

$$q^c = 1.8 * q_{ult} \left[ \frac{z_{50}}{z_{50} + 50 * |z_0^+ - z^g|} - \frac{z_{50}}{z_{50} + 50 * |z_0^- - z^g|} \right] \quad (4.13)$$

where:

$q^d$  = Tensile resistance of the qz spring.

$C_d$  = ratio of the maximum drag (suction) force to the ultimate resistance of the q-z material.

$q^c$  = Closure force of the spring.

#### *Soil vertical stiffness*

Spring beneath the foundation must be properly calibrated to reflect the actual initial stiffness of the soil by using the impedance functions described in section 2.2. Initial vertical stiffness depends of foundation geometry, shear modulus and Poisson ratio of the soil. In addition, Gazetas et Stokoe (1991) suggests a modification factor to account for the effect of the embedment of the foundation into the soil. Since further deep the footing is founded, a stiffer effect may have, the embedment factor provides a factor greater than one.

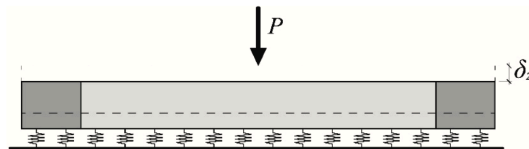


Figure 4.20 Distributed springs with vertical stiffness. Adapted from ASCE41 (2013).

$$K'_z = \frac{GL}{1 - \nu} \left[ 0.73 + 1.54 \left( \frac{B}{L} \right)^{0.75} \right] \quad (4.14)$$

$$\eta_z = \left[ 1 + \frac{D}{21 * B} \left( 1 + 1.3 \frac{B}{L} \right) \right] \left[ 1 + 0.2 \left( \frac{A_w}{4BL} \right)^{2/3} \right] \quad (4.15)$$

where:

$G$  = Effective shear modulus. It can be determined as the product initial shear Modulus  $G_0$  and the reduction factor on Table 19.3-2 of ASCE7-16.

$\nu$  = Poisson ratio of the soil. Suggested as 0.25 for sands

$L$  = Half footing Length

$B$  = Half footing width

$D$  = Embedment deepness

$A_w$  = Lateral contact area of the foundation to the soil along the perimeter of the footing

$\eta_z$  = Modification factor of vertical stiffness due to the embedment effect of the footing. It shall be noted that  $e_z$  terminology is used as well in Figure 2.34.

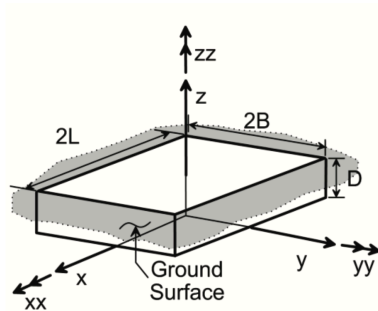


Figure 4.21 Footing geometric parameters. Adapted from NEHRP (2012).

### Soil rotational stiffness

In order to describe the rotational movements of the foundation, spring's stiffness need to be appropriately selected. The methodology suggested in the literature (NEHRP 2012) involves two key parameters, (1) a designated end sections along the length of the foundation expressed as ratio of the half overall length,  $R_e$ , and (2) an increase of the spring's stiffness placed inside this end region expressed as a parentage of the stiffness inner springs,  $R_k$ .

As it can be seen in Figure 4.22, the stiffness of individual springs located at the end regions is in function of its unitary vertical stiffness, its tributary area and the  $R_k$  factor. The following equation used to estimate  $R_k$  factor, was derived by matching the moment produced by the springs for a unit foundation rotation and for a given  $R_e$  ratio (typically taken between 0.3 and 0.5). It should be noted that  $R_e$  ratio is estimated with half of the footing length and that the edge springs have half of the calculated stiffness since they have half of the tributary area.

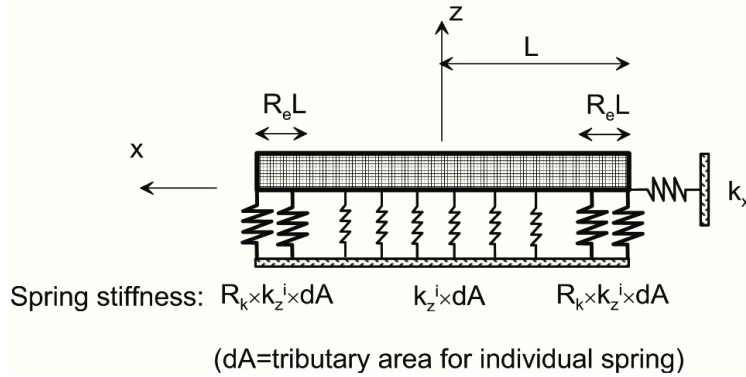


Figure 4.22 BNWF model with stiffer springs at the end regions. Adapted from NEHRP (2012).

$$k_z^i = \frac{k_z}{4BL} \quad (4.16)$$

$$k_{yy,surface} = \frac{G}{1-\nu} (I_y)^{0.75} \left[ 3 \left( \frac{L}{B} \right)^{0.15} \right] \quad (4.17)$$

$$\eta_{yy} = 1 + 0.92 \left( \frac{d_w}{B} \right)^{0.6} \left[ 1.5 + \left( \frac{d_w}{D} \right)^{1.9} \left( \frac{B}{L} \right)^{-0.6} \right] \quad (4.18)$$

$$k_{yy} = k_{yy,surface} * \eta_{yy} \quad (4.19)$$

$$R_e = \frac{L_{end}}{L} \quad (4.20)$$

$$R_{k,yy} = \frac{\left( \frac{3k_{yy}}{4k_z^i B L^3} \right) - (1 - R_e)^3}{1 - (1 - R_e)^3} \quad (4.21)$$

where:

$k_z$  = vertical stiffness of the overall foundation

$k_z^i$  = vertical stiffness of a single spring

$k_{yy,surface}$  = rotational stiffness of the footing about the y axis at surface level

$\eta_{yy}$  = Modification factor of rotational stiffness about y axis due to the embedment effect of the footing.

$k_{yy}$  = rotational stiffness of the footing about the y axis corrected by embedment effect.

$I_y$  = moment of inertial about y axis of the contact area between the soil and the footing.

$d_w$  = footing thickness.

$D$  = Foundation deepness.

### *Spring spacing*

Sensitivity studies performed by Raychowdhury (2008) have shown than the number of springs and their spacing have an important impact on the response. For instance, the greater number of springs used, the smoother the moment-rotation curves since settlements and internal springs forces

are better distributed along the length of the footing. Furthermore, a coarse distribution of springs has been associated with an important increase in the settlements. Raychowdhury (2008) show that the results obtained for springs (vertical displacements - settlements and internal forces) tend to converge when a minimum of 25 springs, with a spacing of 4% of the overall length of the foundation. These suggestions were adopted in the present study.

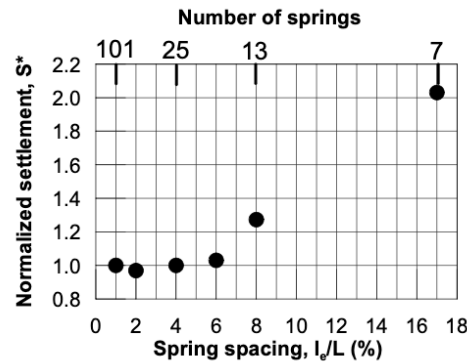


Figure 4.23 Effect of spring spacing on normalized settlement demand. Adapted from Raychowdhury (2008).

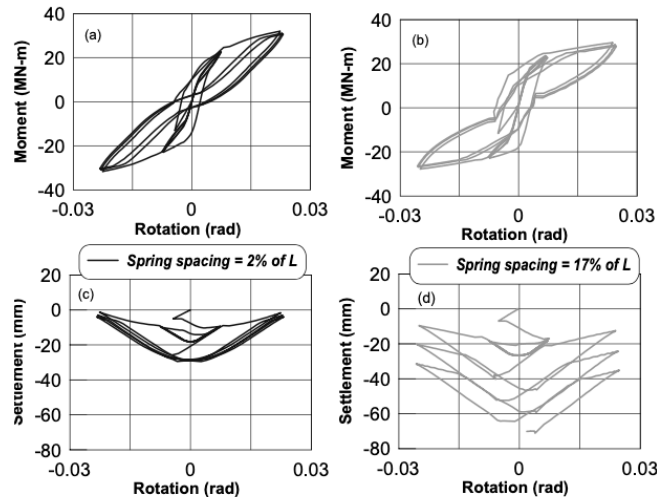


Figure 4.24 Effect of spring spacing on overall footing response: (a)–(b) moment versus rotation and (c)–(d) settlement versus rotation. Adapted from Raychowdhury (2008).

### *Soil damping*

The methodology to implement BNWF in OpenSees allows to explicitly incorporate the effect of the radial damping due to footing rocking behaviour using the following impedance function

proposed by Pais and Kausel (1988). Hysterical damping is introduced to the structural system by the material inelastic constitutive law through energy dissipated by plastic deformation.

$$\beta_z = \left[ \frac{4 \left[ \psi \left( \frac{L}{B} \right) + \left( \frac{D}{B} \right) \left( 1 + \frac{L}{B} \right) \right]}{\left( \frac{k_z}{GB} \right)} \right] \left[ \frac{a_0}{2\alpha_z} \right] \quad (4.22)$$

$$\alpha_0 = \frac{\omega B}{V_s} \quad (4.23)$$

$$\psi = \sqrt{\frac{2(1-\nu)}{(1-2\nu)}} \leq 2.5 \quad (4.24)$$

$$\alpha_z = 1 - \left[ \frac{\left( 0.4 + \frac{0.2}{L/B} \right) a_0^2}{\left( \frac{10}{1 + 3(L/B - 1) + a_0^2} \right)} \right] \quad (4.26)$$

where

$\beta_z$  = Vertical radial damping

$a_0$  = Dimensionless factor

$\omega$  = Natural frequency of the structure

$\psi$  = Dimensionless factor

$\alpha_z$  = Stiffness modification factor

#### *Soil tension capacity*

The QzSimple material allows the user to consider the tensile resistance of the soil-foundation interface as percentage of the compressive resistance. However, in most cases, tensile resistance is neglected in order to consider the most unfavorable scenario when uplift due to the foundation is triggered. Hamid (2023) have shown that a small amount of tensile resistance can be helpful to achieve analysis convergence when very soft soils are considered.

The specific meshing related firstly parameters are presented in Table 4.12. Since parametric tests performed by Raychowdhury and Hutchinson (2008) concluded that a minimum of 25 springs seems reasonable enough to achieve analysis convergence for vertical settlements, 31 springs for all studied scenarios were selected. A constant distribution of 31 springs along the overall length of the footings results in a spacing of 3.3% the total length. The expensive literature about this matter recommends at least a spacing of 4%, therefore the selected criteria seems appropriate. Since a uniform spring spacing,  $S_{sp}$ , is chosen, the way in which the rotational stiffness of the footing is added is by increase the springs stiffens by a rate of  $R_k$  within the Lend region of the footing.  $R_k$  ratio is estimated with equation 4.21 and it depends on the ratio of the end region and the overall length of the footing,  $R_e$ . Following the recommendation of NEHRP (2012) and seeking for consistency with Murugananthan (2018) study, the  $R_e$  ration used throughout all scenarios remain constant with a value of  $0.40L_f$  of the total length of the footing ( $0.20$  each  $L_{end}$ ).

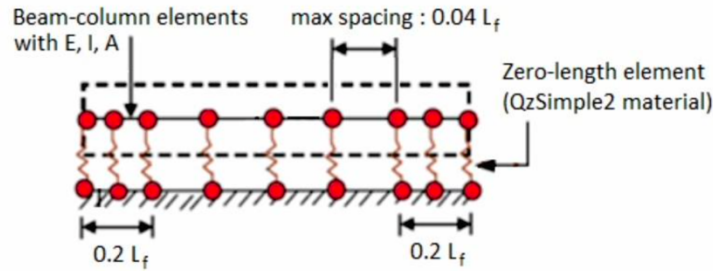


Figure 4.25 System Soil-Column-Foundation. OpenSees modeling of the system. Adapted from Raychowdhury et Hutchinson (2008).

Table 4.12 Footing beam-column element meshing related parameters

| Scenario           | $S_{sp}$<br>(mm) | N<br>(Qty.) | $L_{end}$<br>(mm) | $R_k$ | $R_e$ |
|--------------------|------------------|-------------|-------------------|-------|-------|
| 6S-MTL-NBC20-E-NCP | 666.67           | 31          | 4000              | 2.5   | 0.40  |
| 6S-MTL-NBC20-E-CP  | 700.00           | 31          | 4200              | 2.5   | 0.40  |
| 6S-MTL-NBC20-C-NCP | 466.67           | 31          | 2800              | 2.3   | 0.40  |
| 6S-MTL-NBC20-C-CP  | 491.67           | 31          | 2950              | 2.4   | 0.40  |
| 6S-VCR-NBC20-E-NCP | 733.33           | 31          | 4400              | 2.5   | 0.40  |
| 6S-VCR-NBC20-E-CP  | 766.67           | 31          | 4600              | 2.5   | 0.40  |
| 6S-VCR-NBC20-C-NCP | 533.33           | 31          | 3200              | 2.1   | 0.40  |
| 6S-VCR-NBC20-C-CP  | 600.00           | 31          | 3600              | 2.0   | 0.40  |
| 3S-MTL-NBC20-E-NCP | 550.00           | 31          | 3300              | 2.4   | 0.40  |
| 3S-MTL-NBC20-E-CP  | 600.00           | 31          | 3600              | 2.3   | 0.40  |
| 3S-MTL-NBC20-C-NCP | 383.33           | 31          | 2300              | 2.4   | 0.40  |
| 3S-MTL-NBC20-C-CP  | 416.67           | 31          | 2500              | 2.5   | 0.40  |
| 3S-VCR-NBC20-E-NCP | 533.33           | 31          | 3200              | 2.4   | 0.40  |
| 3S-VCR-NBC20-E-CP  | 583.33           | 31          | 3500              | 2.4   | 0.40  |
| 3S-VCR-NBC20-C-NCP | 433.33           | 31          | 2600              | 2.2   | 0.40  |
| 3S-VCR-NBC20-C-CP  | 466.67           | 31          | 2800              | 2.2   | 0.40  |

Secondly, the spring response related parameters are shown in Table 4.13. The springs ultimate capacity is easily calculated as the ultimate soil bearing capacity indicated in 3.6.1 and multiplied by the tributary area of each springs. It should be noticed that the springs placed at the edge of the footing, capacity,  $q_{ult}$ , shall be modified by half since they carry out half tributary area of the rest of the springs. On the other hand,  $z_{50}$  for each region, can be calculated with equation 4.11 once  $q_{ult}$  initial stiffness,  $k_{in}$ , are estimated. It is remarked that  $k_{in}$  for springs located at the end regions are amplified by  $R_k$  factor.

Table 4.13 Spring response related parameters

| Scenario           | Q <sub>ult</sub><br>(N) | Q <sub>ult,edge</sub><br>(N) | Z <sub>50,end</sub><br>(mm) | Z <sub>50,mid</sub><br>(mm) | $\beta_z$ |
|--------------------|-------------------------|------------------------------|-----------------------------|-----------------------------|-----------|
| 6S-MTL-NBC20-E-NCP | 2266667                 | 1133333                      | 40.20                       | 100.14                      | 0.14      |
| 6S-MTL-NBC20-E-CP  | 2520000                 | 1260000                      | 42.34                       | 105.68                      | 0.15      |
| 6S-MTL-NBC20-C-NCP | 6300000                 | 3150000                      | 38.02                       | 88.30                       | 0.03      |
| 6S-MTL-NBC20-C-CP  | 7375000                 | 3687500                      | 40.86                       | 96.26                       | 0.03      |
| 6S-VCR-NBC20-E-NCP | 2786667                 | 1393333                      | 44.49                       | 111.22                      | 0.29      |
| 6S-VCR-NBC20-E-CP  | 3066667                 | 1533333                      | 46.63                       | 116.75                      | 0.31      |
| 6S-VCR-NBC20-C-NCP | 6000000                 | 3000000                      | 38.70                       | 81.76                       | 0.04      |
| 6S-VCR-NBC20-C-CP  | 6750000                 | 3375000                      | 41.72                       | 84.73                       | 0.04      |
| 3S-MTL-NBC20-E-NCP | 1320000                 | 660000                       | 31.28                       | 75.04                       | 0.21      |
| 3S-MTL-NBC20-E-CP  | 1440000                 | 720000                       | 33.03                       | 77.48                       | 0.21      |
| 3S-MTL-NBC20-C-NCP | 5175000                 | 2587500                      | 33.61                       | 82.12                       | 0.05      |
| 3S-MTL-NBC20-C-CP  | 6875000                 | 3437500                      | 38.18                       | 95.86                       | 0.06      |
| 3S-VCR-NBC20-E-NCP | 1280000                 | 640000                       | 30.68                       | 74.18                       | 0.29      |
| 3S-VCR-NBC20-E-CP  | 1400000                 | 700000                       | 32.45                       | 76.69                       | 0.30      |
| 3S-VCR-NBC20-C-NCP | 4420000                 | 2210000                      | 32.72                       | 71.58                       | 0.06      |
| 3S-VCR-NBC20-C-CP  | 4900000                 | 2450000                      | 34.66                       | 74.77                       | 0.06      |

## 4.4 Model limitations

Although quite appropriate, the presented modelling methodology has certain limitation, namely:

- Fracture of the diagonal sections cannot be represented.
- Local buckling of the diagonal walls due to their slenderness is not taken into account in the fiber cross section formulation.
- Yielding of beams and columns section is not captured since they are modelled as elastic elements.
- Shear deformations are not explicitly introduced since they are not expected to have a significant impact on the response.
- Lateral sliding of the foundation is not explicitly considered.

## 4.5 Ground motions

Successful investigation of seismic response of building structures heavily depend on (i) appropriate representation of behaviour of structural components and materials and (ii) appropriate ground motion input. In this section the procedure applied to constitute ground motion datasets is presented along with the description of scaling procedures employed.

Ground motions, GM, are acceleration records of historical or synthetic earthquakes. They are essential for any time-history analysis of structures as they represent earthquake loading. The selected ground motion records shall be representative of the seismic hazard at studied location and site conditions. Different methods have been proposed to select appropriate ground motion records, and to scale them to correspond to design spectra used to conceive SFRS. Selecting “good” ground motion candidates is a complex task. In this study a methodology proposed in the NBC 2015 guidelines for time-history analysis was followed. Unfortunately, at the moment of writing this document the guidelines for 2020 edition were not yet available for the public, however invaluable comments and suggestions from some members of the committee responsible for their publication were available (Tremblay, 2023) and were taken into account to constitute ground motion ensembles for the analysis.

Some of the important points discussed in NBC 2015 guidelines are summarized below.

Seismic hazard deaggregation analysis at the design location allows an insight into the regional seismicity and identify the type of earthquake that contribute significantly (example crustal, in-slab and interface in western Canada). In addition, it is also possible to identify M-R scenarios that contribute the most significantly to seismic hazard.

Historical ground motions recorded on similar site conditions are favoured. It is strongly suggested to select a GM recorded on sites with similar ground conditions as the design site of interest.

Synthetic ground motions generated using a methodology that reflect seismo-geotechnical conditions of design region can be used, independently or to complement historical ground motion records.

Scaling is the process in which the response spectra of the selected GM candidates are compared to a target spectrum, most often defined by the local building code, and scaled up or down to match the target spectrum. At the end of the process, the GMs whose response spectra fits naturally the target spectrum the best are chosen to perform the time-history analysis.

Figure 4.26 shows the occurrence of earthquake records of various magnitudes in western and eastern Canada. The seismicity of western Canada is well known and large variety of historical ground motions is available. Since Pacific coast lies on the so-called *Ring of fire*, high tectonic and volcanic activity is seen in the region. On the other hand, the seismicity of Eastern Canada, is more complex and small number of historical recording is available. The Canada Geological Survey developed the deaggregation plots for several locations across the country compatible with the seismic hazard model used to derive NBCC 2015 seismic hazard maps. They show the contribution to hazard for a diverse range of MR scenarios, allowing to rapidly determine the importance of the contribution of each scenario to the total hazard. Plots are presented for spectral accelerations at various periods, which is a convenient way to visualize which different earthquake sources can trigger a specific range of wave frequencies affecting different spectral periods.

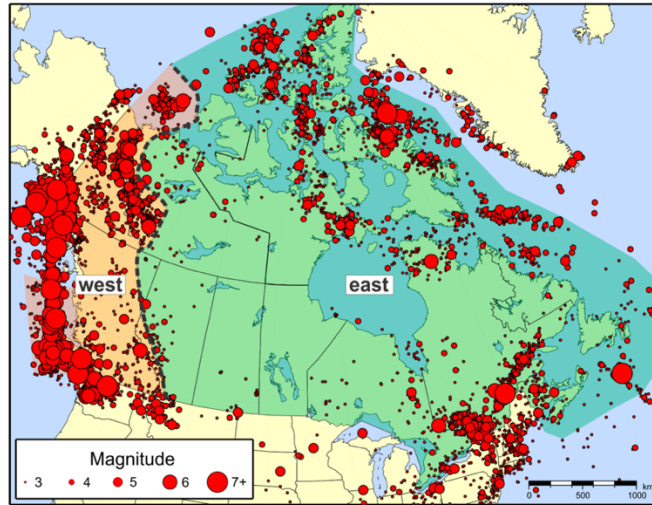


Figure 4.26 Map of Canada showing the earthquake catalog with dashed lines delimiting the eastern and western seismic regions. Adapted from Adams et al. (2019).

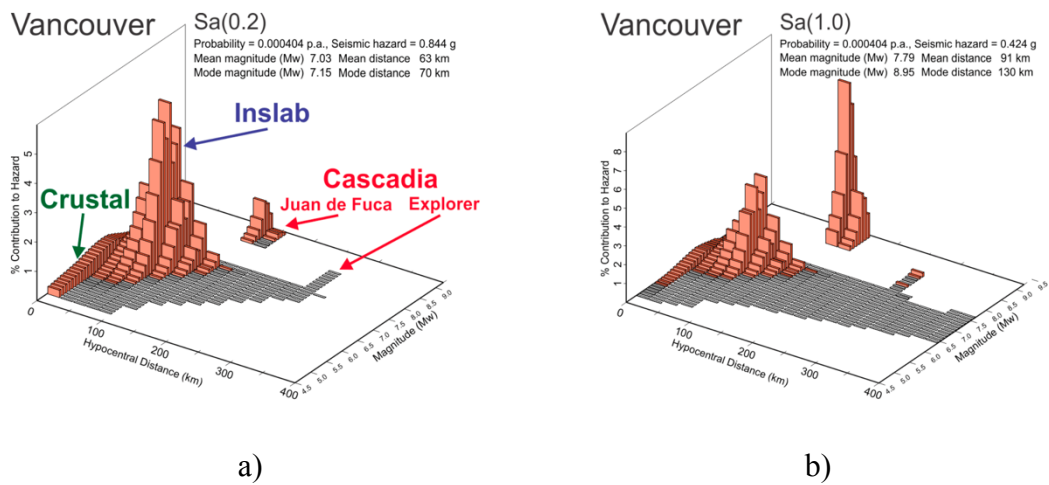


Figure 4.27 Deaggregation plot for Vancouver. a) Spectral acceleration at 0.2 s, b) Spectral acceleration at 1.0 s. Adams et al. (2019)

It can be seen from Figure 4.27 that for Vancouver short periods, hazard contribution of *in-slab* earthquakes is significantly higher than *crustal* or *interface* (Juan de Fuca subduction zone) while for period of 1 second, the interface type, characterized by trigger lowest frequencies, contributes more. With this information, a mean magnitude and the mean distance for each deaggreaction chart is calculated with the purpose of summarized all contribution at each given period.

Table 4.14 Mean magnitude ( $M_w$ ) and Mean Distance R (km) for hazard contribution for Montreal and Vancouver

| Modal period<br>(s) | Montreal |        | Vancouver |        |
|---------------------|----------|--------|-----------|--------|
|                     | $M_w$    | R (km) | $M_w$     | R (km) |
| 0.1                 | 6.60     | 27     | 6.89      | 58     |
| 0.2                 | 6.51     | 29     | 7.03      | 63     |
| 0.3                 | 6.59     | 32     | 7.16      | 68     |
| 0.5                 | 6.69     | 37     | 7.33      | 75     |
| 1                   | 6.86     | 50     | 7.79      | 91     |
| 2                   | 7.00     | 67     | 8.12      | 107    |
| 5                   | 7.17     | 88     | 8.68      | 131    |
| 10                  | 7.25     | 111    | 8.74      | 134    |

Hazard level: 0.000404 per annum (2% of exceed in 50 yrs)

Soil Class: C

Shaded for periods of interest for this study

Mean distance and mean magnitude calculated for different spectral periods allows to gather ground motions from a variety of databases and constitute the appropriate sets for the analyses. For “western North-America” seismic conditions, there are many databases that contains GM records. Among the most used in North America are the PEER *NGA-WEST2 DATABASE* maintained by the *Pacific Earthquake Engineering Research Center* which contain shallow earthquakes records (i.e. crustal and in-slab events) and the Japanese databases *K-NET* and *KiK-NET* providing shallows and interface records. To select ground motion ensembles for Vancouver building, using additional databases were also investigated. However, no records were considered as appropriated for this study because the followed reasons:

Mexican database *RAII-UNAM*. Since Mexican seismic hazard models are referenced to site classified as rock, most of the GM provided in this database for the M-R scenarios needed by this database were recorded directly on rock sites.

New Zealand database *GeoNet* provides 756 pairs (NS and EW) of GMs suitable for structural time-history analyses. Nevertheless, after scaling the records to match with the target spectrum, all of them required a scale factor beyond the limits suggested by the NBC 2015 guidelines (specific scale factor from 0.2 to 5, seems appropriate values).

Canadian synthetic ground motions generated by Atkinson (2009) available via *Engineering Seismology Toolbox* were dismissed following the recommendation of one member of the committee responsible for the NBC Guidelines 2020 as it has been observed that the methodology followed in the creation of this database does provides sufficient variability between ground motion records. (Tremblay, 2023). In addition, Atkinson's database was created to match NBCC 2005 design spectra.

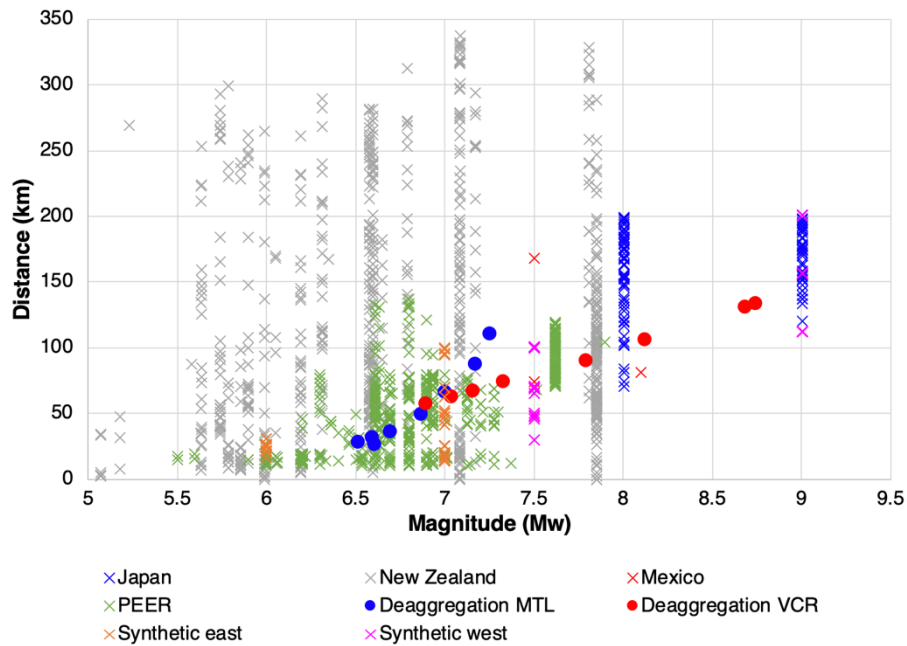


Figure 4.28 Gathered ground motions records

Individual GM candidates are shown as crosses in Figure 4.28, in function of the mean distance and mean magnitude. The colour of the crosses identifies a specific database. Most of the gathered GM are from *crustal* or *in-slab* group of earthquakes. From the historical group of records, only Japanese earthquakes were from *interface* type, notably from Tohoku event (2011) with a moment magnitude of 9.1, and from Tokachi-Oki event (2003) with a moment magnitude of 8.3. It should be noted that each GM shown in Figure 4.28 represents actually a pair of horizontal ground motion components simultaneously in two perpendicular directions. This pairs are statistical coupled and are commonly used in GM sets for 3D analysis. Since a 2D analysis was conducted in this study,

two components were treated separately to increase the number of candidates, but during selection process the attention was paid to avoid select of both components whenever possible.

Because the lack of a robust database that contain historical GM recorded in Eastern Canada or in regions with similarly seismological mechanisms and the limitation of the available time given to complete this study, it was decided to investigate only the frame response for Vancouver scenarios. Future work will be consecrated for the construction of a proper dataset in order extend the scope of this project to Montreal scenarios as well.

Once the set of GMs candidates is formed, final selection and scaling can be made. The methodology is supported by the fact that a given response spectrum preserves its form when ground motion accelerations are amplified or diminished by a constant. Consequently, if the response spectrum is scaled, the corresponding ground motion accelerogram will be scaled at the same rate. This characteristic permits to match response spectrums to a desired target spectrum since the seismic hazard is described by spectral acceleration and not only by peak values, such as PGA or PGV. The procedure implemented in this study to select and scale GM that is described below follows the methodology provided in the Commentary J of NBC 2015 proposed by Tremblay et al. (2015) and can be summarized as follows.

The target spectrum is established first. This spectrum represents the hazard level on the location and site of interest. It can be taken as the design spectrum with adjusted values in the short period range to reflect better the actual nature of a given response spectrum.

The response spectrum of GMs candidate is computed with the aid of a Matlab code developed within the scope of this study in order to automatically compute the response spectra for the large number of candidate GMs. The period increments in which the response spectrum was calculated were linearly distributed. *Exact numerical integration with linear segments* at 5% of critical damping was implemented for the computations.

It is suggested to narrow the period range for which the scaling is conducted to achieve a better fit between the target and response spectrum of candidate GM by considering only those portions of the response spectrum that influence the structural response. The lower end of this period range aiming to capture the response influenced by the higher modes is taken as the minimum between  $0.20 T_1$  and the period needed to accumulate the 90% of the participative mass,  $T_{90\%}$ . The upper limit of the range is the larger value between  $2.0 T_1$  and 1.5s with the objective of capture the period

lengthening that may occur during inelastic response. A range period between 0.1 s and 1.6 s was judged appropriate for all Vancouver scenarios (sites Classes E or C , 3- of 6-storeys height).

The scale factor is applied at each period increment within the range of periods of interest, to match the spectral accelerations of the repose spectrum of a GM and the target spectrum. The average value of all scale factors within the period range considered is taken as the scaling factor for that particular ground motion. The value of scaling factor should be kept between 0.2 and 5.0 to avoid significant modifications of accelerations of the original ground motion.

The selection methodology aims to favour GMs with response spectra that naturally fit better the target spectrum over the selected range of periods. The standard deviation is calculated for each individual ground motion and those with the lowest standard deviation are retained.

The NBC 2015 guidelines suggest 11 as the minim number of GMs for one GM ensemble, with at least 5 GM for each M-R scenario and type of earthquake. This seems sufficient when time-history analysis is used for design. Because the goal of this study was to study behaviour, validate design procedures and eventually propose a revision to the design portions, it was suggested (Tremblay 2023) to at least duplicate the number of selected GM. It is also suggested to regroup crustal and in-slab events into the same GM subgroup and interface GM into another. Lastly, 30 GMs records were chosen for each M-R scenario.

In the final step, a verification is made to ensure that the average of the all scaled response spectrum do not fall 10% below the target spectrum. If this requirement is not meet, an additional unique scale factor need to be applied to all GM that form the ensemble. During this process, it had not need to apply this additional factor.

Response spectra of unscaled and scaled ground motions, selected for Vancouver Site Class C and E, along with the mean spectrum and design spectrum are shown in Figures 4.29 and 4.31 respectively.

M-R scenarios of the final selection of GM are shown in figures 4.30 and 4.32. In Tables 4.15 and 4.16 present detailed information about the final selection for site Class C and site Class E respectively.

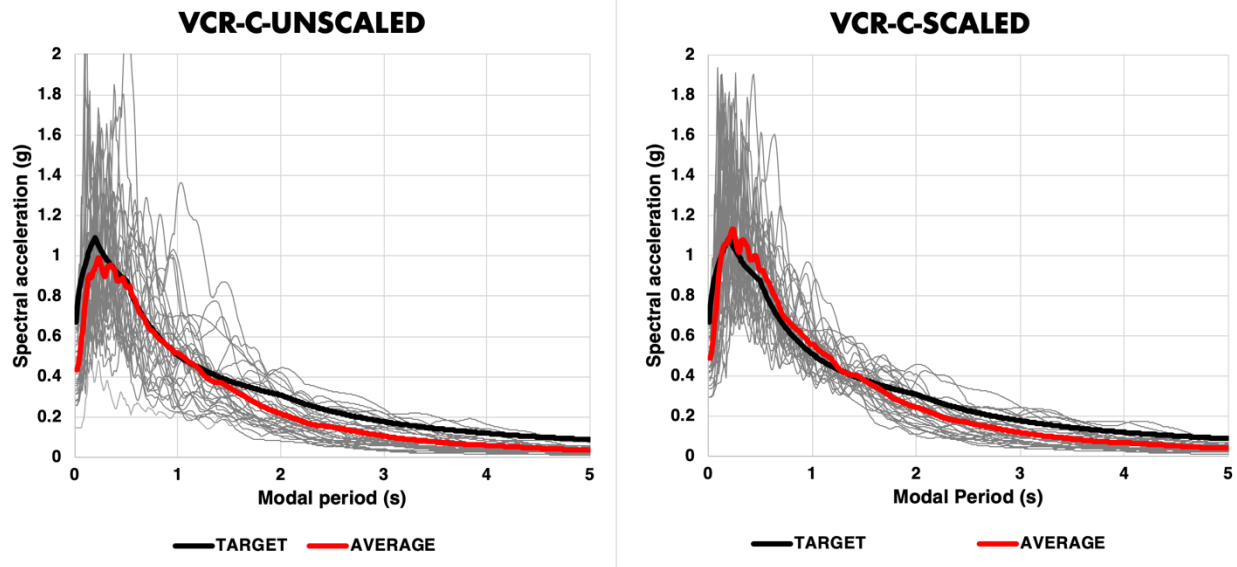


Figure 4.29 Unscaled and scaled ground motions for Vancouver site Class C scenarios.

Table 4.15 Selected ground motions for Vancouver site Class C scenarios.

| Record ID | Scale factor | Record                            | Event type | Earthquake Name         | Year | Mw   | R (km) | Site Class |
|-----------|--------------|-----------------------------------|------------|-------------------------|------|------|--------|------------|
| 438       | 1.04         | RSN3746_CAPEM<br>END_CBF360.AT2   | Crustal    | "Cape Mendocino"        | 1992 | 7.01 | 16.44  | C          |
| 412       | 1.07         | RSN1633_MANJIL<br>ABBAR--T.AT2    | Crustal    | "Manjil_ Iran"          | 1990 | 7.37 | 12.55  | C          |
| 378       | 2.04         | RSN801_LOMAP_<br>SJTE315.AT2      | Crustal    | "Loma Prieta"           | 1989 | 6.93 | 14.18  | C          |
| 343       | 1.06         | RSN265_VICT_CP<br>E045.AT2        | Crustal    | "Victoria_<br>Mexico"   | 1980 | 6.33 | 13.8   | C          |
| 506       | 1.66         | RSN5478_IWATE_<br>AKT023EW.AT2    | Crustal    | "Iwate_ Japan"          | 2008 | 6.90 | 11.68  | C          |
| 515       | 1.27         | RSN5664_IWATE_<br>MYG005NS.AT2    | Crustal    | "Iwate_ Japan"          | 2008 | 6.90 | 10.71  | C          |
| 425       | 1.01         | RSN2627_CHICHI.<br>03_TCU076E.AT2 | Crustal    | "Chi-Chi_<br>Taiwan-03" | 1999 | 6.20 | 13.04  | C          |
| 488       | 0.96         | RSN4850_CHUETS<br>U_65013EW.AT2   | Crustal    | "Chuetsu-oki_<br>Japan" | 2007 | 6.80 | 13.68  | C          |
| 439       | 0.90         | RSN3748_CAPEM<br>END_FFS270.AT2   | Crustal    | "Cape Mendocino"        | 1992 | 7.01 | 16.64  | C          |

Table 4.15 Selected ground motions for Vancouver site Class C scenarios (continue).

|       |      |                              |           |                     |      |      |       |   |
|-------|------|------------------------------|-----------|---------------------|------|------|-------|---|
| 374   | 1.33 | RSN755_LOMAP_CYC285.AT2      | Crustal   | "Loma Prieta"       | 1989 | 6.93 | 19.97 | C |
| 489   | 1.23 | RSN4850_CHUETSU_65013NS.AT2  | Crustal   | "Chuetsu-oki Japan" | 2007 | 6.80 | 13.68 | C |
| RBT1  | 1.42 | 103156ol_y0a_Scaled.acc      | Inslab    | USA, Olympia        | 1949 | 6.70 | 50    | C |
| RBT2  | 1.70 | 103156ol_y0b_Scaled.acc      | Inslab    | USA, Olympia        | 1949 | 6.70 | 50    | C |
| RBT5  | 1.54 | TE01001u.smc.F1.a Scaled.acc | Inslab    | El Salvador         | 2001 | 7.70 | 81    | C |
| RBT3  | 1.85 | 0725a a Scaled.acc           | Inslab    | USA, Nisqually      | 2001 | 6.93 | 20.34 | C |
| JP83  | 0.70 | MYG0111103111446.EW          | Interface | TOHOKU 2011         | 2011 | 9.00 | 120   | B |
| JP235 | 0.47 | HKD1130309260450.EW          | Interface | TOKACHI-OKI 2003    | 2003 | 8.00 | 74    | C |
| JP9   | 0.78 | FKS0071103111446.EW          | Interface | TOHOKU 2011         | 2011 | 9.00 | 184   | C |
| JP68  | 0.48 | MYG0021103111446.NS          | Interface | TOHOKU 2011         | 2011 | 9.00 | 137   | C |
| JP264 | 1.73 | KSRH050309260450.NS2         | Interface | TOKACHI-OKI 2003    | 2003 | 8.00 | 164   | C |
| JP231 | 1.09 | HKD1110309260450.EW          | Interface | TOKACHI-OKI 2003    | 2003 | 8.00 | 81    | C |
| JP94  | 1.01 | MYG0161103111446.NS          | Interface | TOHOKU 2011         | 2011 | 9.00 | 197   | C |
| JP243 | 0.38 | KKWH080309260450.NS1         | Interface | TOKACHI-OKI 2003    | 2003 | 8.00 | 182   | D |
| JP228 | 1.02 | HKD1090309260450.NS          | Interface | TOKACHI-OKI 2003    | 2003 | 8.00 | 117   | C |
| JP232 | 1.06 | HKD1110309260450.NS          | Interface | TOKACHI-OKI 2003    | 2003 | 8.00 | 81    | C |
| JP52  | 1.35 | IWTH221103111446.NS2         | Interface | TOHOKU 2011         | 2011 | 9.00 | 192   | C |
| JP56  | 0.82 | IWTH231103111446.NS2         | Interface | TOHOKU 2011         | 2011 | 9.00 | 158   | B |
| JP227 | 1.58 | HKD1090309260450.EW          | Interface | TOKACHI-OKI 2003    | 2003 | 8.00 | 117   | C |
| JP245 | 1.16 | KSRH010309260450.EW1         | Interface | TOKACHI-OKI 2003    | 2003 | 8.00 | 184   | C |
| JP258 | 1.23 | KSRH040309260450.EW2         | Interface | TOKACHI-OKI 2003    | 2003 | 8.00 | 167   | D |

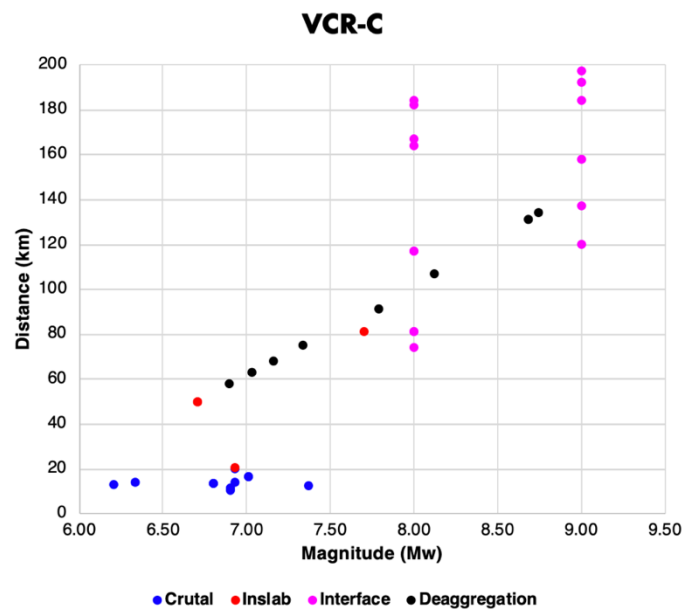


Figure 4.30 Selected ground motions for Vancouver Class C site scenarios.

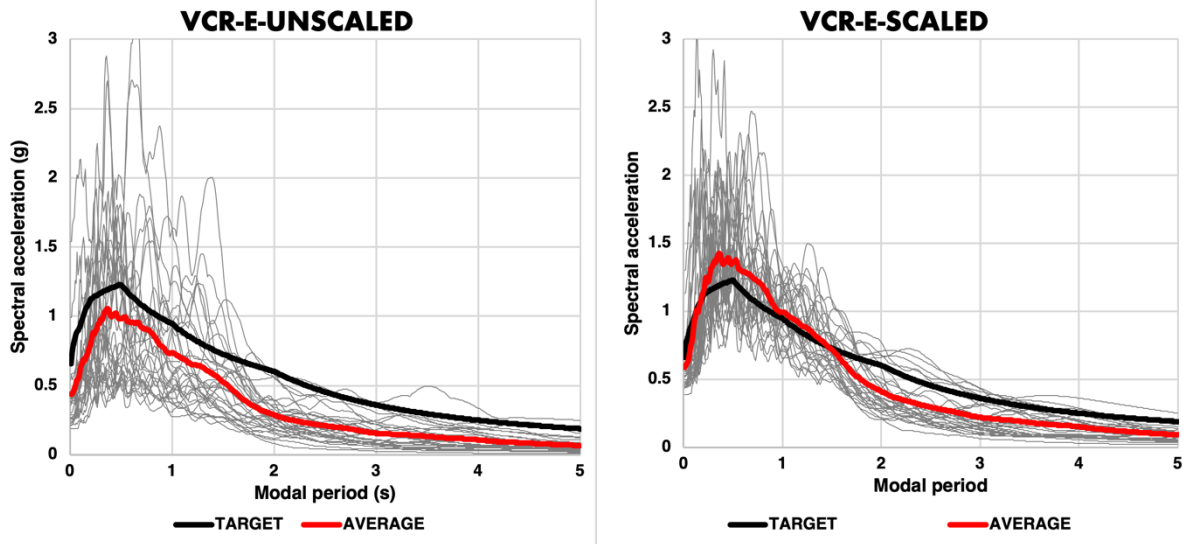


Figure 4.31 Unscaled and scaled ground motions for Vancouver site Class E scenarios.

Table 4.16 Selected ground motions for Vancouver site Class E scenarios.

| Record ID | Scale factor | Record                        | Event type | Earthquake Name         | Year | Mw   | R (km) | Site Class |
|-----------|--------------|-------------------------------|------------|-------------------------|------|------|--------|------------|
| 30        | 2.06         | RSN728_SUPER.B_B-WSM180.AT2   | Crustal    | "Superstition Hills-02" | 1987 | 6.54 | 13.03  | E          |
| 27        | 2.03         | RSN721_SUPER.B_B-ICC000.AT2   | Crustal    | "Superstition Hills-02" | 1987 | 6.54 | 18.2   | E          |
| 50        | 2.07         | RSN1147_KOCAELI_ATS090.AT2    | Crustal    | "Kocaeli_Turkey"        | 1999 | 7.51 | 68.09  | E          |
| 205       | 2.43         | RSN6890_DARFIELD_CMHSN10E.AT2 | Crustal    | "Darfield_New Zealand"  | 2010 | 7.00 | 17.64  | E          |
| 207       | 1.96         | RSN6959_DARFIELD_REHSN02E.AT2 | Crustal    | "Darfield_New Zealand"  | 2010 | 7.00 | 19.48  | E          |
| 38        | 1.73         | RSN759_LOMAP_A01090.AT2       | Crustal    | "Loma Prieta"           | 1989 | 6.93 | 43.77  | E          |
| 35        | 1.78         | RSN758_LOMAP_EM Y260.AT2      | Crustal    | "Loma Prieta"           | 1989 | 6.93 | 76.87  | E          |
| 36        | 2.60         | RSN758_LOMAP_EM Y350.AT2      | Crustal    | "Loma Prieta"           | 1989 | 6.93 | 76.87  | E          |
| 11        | 2.40         | RSN178_IMPVAL.LH H-E03140.AT2 | Crustal    | "Imperial Valley-06"    | 1979 | 6.53 | 10.79  | E          |
| 37        | 2.32         | RSN759_LOMAP_A01000.AT2       | Crustal    | "Loma Prieta"           | 1989 | 6.93 | 43.77  | E          |

Table 4.16 Selected ground motions for Vancouver site Class E scenarios (continue).

|       |      |                                  |           |                            |      |      |       |   |
|-------|------|----------------------------------|-----------|----------------------------|------|------|-------|---|
| 208   | 2.19 | RSN6959_DARFIELD<br>REHSS88E.AT2 | Crustal   | "Darfield_ New<br>Zealand" | 2010 | 7.00 | 19.48 | E |
| RBT4  | 1.17 | BA01003u.smc.F1.a_S<br>caled.acc | Inslab    | Unknown                    | unk  | unk  | unk   | E |
| RBT2  | 1.21 | 2147b_a_Scaled.acc               | Inslab    | Unknown                    | unk  | unk  | unk   | E |
| RBT3  | 1.40 | AR01001u.smc.F1.a_S<br>caled.acc | Inslab    | Unknown                    | unk  | unk  | unk   | E |
| RBT1  | 1.48 | 1412c_a_Scaled.acc               | Inslab    | Unknown                    | unk  | unk  | unk   | E |
| JP72  | 0.72 | MYG0041103111446.<br>NS          | Interface | TOHOKU 2011                | 2011 | 9.00 | 175   | E |
| JP71  | 0.56 | MYG0041103111446.<br>EW          | Interface | TOHOKU 2011                | 2011 | 9.00 | 175   | E |
| JP226 | 0.97 | HKD1080309260450.<br>NS          | Interface | TOKACHI-OKI<br>2003        | 2003 | 8.00 | 136   | D |
| JP243 | 1.21 | KKWH080309260450.<br>NS1         | Interface | TOKACHI-OKI<br>2003        | 2003 | 8.00 | 182   | D |
| JP240 | 0.74 | HKD1260309260450.<br>NS          | Interface | TOKACHI-OKI<br>2003        | 2003 | 8.00 | 198   | E |
| JP244 | 1.32 | KKWH080309260450.<br>NS2         | Interface | TOKACHI-OKI<br>2003        | 2003 | 8.00 | 182   | D |
| JP81  | 1.36 | MYG0101103111446.<br>EW          | Interface | TOHOKU 2011                | 2011 | 9.00 | 143   | E |
| JP95  | 2.48 | MYG0171103111446.<br>EW          | Interface | TOHOKU 2011                | 2011 | 9.00 | 183   | E |
| JP82  | 1.07 | MYG0101103111446.<br>NS          | Interface | TOHOKU 2011                | 2011 | 9.00 | 143   | E |
| JP258 | 1.96 | KSRH040309260450.<br>EW2         | Interface | TOKACHI-OKI<br>2003        | 2003 | 8.00 | 167   | D |
| JP229 | 1.93 | HKD1100309260450.<br>EW          | Interface | TOKACHI-OKI<br>2003        | 2003 | 8.00 | 102   | D |
| JP225 | 0.91 | HKD1080309260450.<br>EW          | Interface | TOKACHI-OKI<br>2003        | 2003 | 8.00 | 136   | D |
| JP210 | 1.23 | HKD1000309260450.<br>NS          | Interface | TOKACHI-OKI<br>2003        | 2003 | 8.00 | 84    | D |
| JP239 | 1.32 | HKD1260309260450.<br>EW          | Interface | TOKACHI-OKI<br>2003        | 2003 | 8.00 | 198   | E |
| JP316 | 1.26 | TKCH070309260450.<br>NS2         | Interface | TOKACHI-OKI<br>2003        | 2003 | 8.00 | 123   | E |

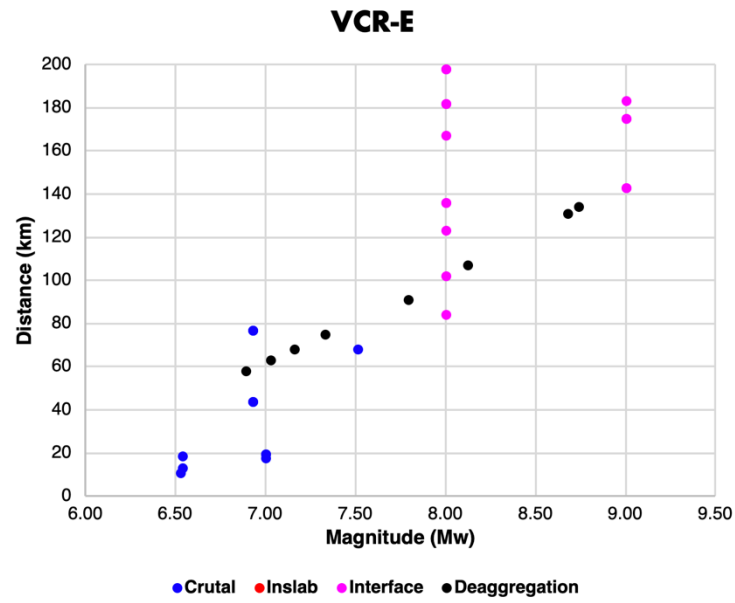


Figure 4.32 Selected ground motions for Vancouver Class E site scenarios.

## 4.6 Analysis protocol

The analysis protocol comprises two stages, vertical loading analysis and the seismic nonlinear time-history analysis arranged in such way that seismic analysis begins from the deformed structure due to vertical loads. By using this sequence, the simulation is able to involve concomitant vertical loading while earthquake occurs and reflect the NBC load combination requirements.

Even if the structure is expected to behave fully elastic under vertical loading, a non-linear analysis is performed in order to involve second order effects. Vertical loading analysis algorithm consist in a non-linear Newton-Raphson algorithm controlled by force with 10 increments. Time-history analysis is solved by the numerical integration procedure Newmark-Beta with the average acceleration variant. This method is well known for its remarkable performance and versatility. It is capable to avoid the problems associated with the numerical damping (amplitude error). Its unconditionally stability, regardless of time increment specified, make it a versatile algorithm. Furthermore, this stability is essential for nonlinear systems. Nevertheless, the time increment shall be at least a tenth of the fundamental period of the analyzed structure, or less, in order to ensure accuracy on the results. Still, most of ground motions accelerograms are recorded with steps of 0.01 s, or lesser, which are often smaller than the suggested limit. As Paultre (2011) strongly suggest, the time increment used for the analysis, match the GM record time increment to exactly represent its discrete series in order to facilitate achieve convergence.

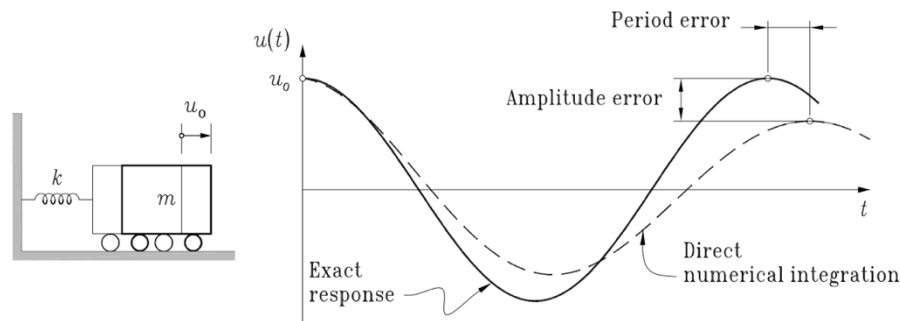


Figure 4.33 Artificial period elongation and amplitude error introduced by direct numerical integration methods. The correct algorithm and calibration are necessary to minimize errors in the response. Adapted from Paultre (2011).

## **CHAPTER 5      RESULTS AND DISCUSION**

In this chapter the results obtained from the nonlinear time-history analyses are presented and discussed. The analyses were conducted for Vancouver building. The response of superstructure-foundation-soil system has been monitored through various response parameters, namely the inter-storey drifts, forces in brace diagonals and plastification occurrences, overturning moments on foundations, foundation vertical displacements, and springs forces. Current design procedure for foundations are critically reviewed in light of these results and opportunities for improvements are discussed.

### **5.1 Analyses description**

The first set of analyses and corresponding results, thereafter referred to as Set 1, are those related to foundations of the frames that do not meet the total inter-storey drift limits, increased because of foundation rotations. A common procedure to meet this requirement is to increase the foundation size and thus diminish the frame drift component induced by foundation rotations. This problem can be also solved by increasing the sizes of the frame members, however, this approach leads to more significant overstrength in the superstructure and has an adverse effect on foundations. As discussed in Chapter 3, for Vancouver, the inter-storey drift limit was a governing criterion for design of both CP and NCP foundations of 3- and 6-storey building. As explained earlier, the previous studies conducted for CBF frames designed according to NBC2015 and S16-14 provisions, showed that inter-storey drifts were well below the limits which motivated the study of these slightly “undersized” foundations. These results serve thus as the baseline to interpret the response and the relevance of the drift provisions related to rotation of foundations. After a careful analysis of obtained results, a second set of analysis has been undertaken (Set 2) to explore opportunities for optimise the size of footing on Class C site and further analyse the response of frames on site Class E.

To further assess the impact of the inclusion of SSI, the analyses are also done for a fixed base model and the results were compared to those obtained from the model that included SSI.

Because the full ground motion set used in the analysis was constituted from two subsets (in-slab and crustal earthquakes in one subset, interface earthquakes in the other), all results presented in

this chapter represent the mean value of five maximum results, as recommended by the NBC 2015 guidelines

Different scenario that were analysed are summarized in Table 5.1. For easier understanding, each scenario (*SC*) was identified by a specific code. A complete designation for each scenario is identified by the following parameters:

Number of storeys: *6S* for six-storey building and *3S* for three-storey building.

Version of the code: *NBC20* for the 2020 version of the National Building Code.

Location: *VCR* for Vancouver location

Site Class: *C* and *E*

Foundation type: *CP* for capacity protected, *NCP* for non-capacity protected, *RdRo=3.9* for footing designed without any capacity design consideration, *blank* form fixed base model (no SSI considerate).

Inter-storey drift requirements related to footing rotations: *A23* for foundation dimensioned in such way to meet fully A23.3 provisions, and *blank* for footings where increase of dimensions to meet the provisions was omitted.

Table 5.1 Considered scenarios

| Scenario                | Run | Set |
|-------------------------|-----|-----|
| SC07-6S-NBC20-VCR-C     | Y   | 1   |
| SC08-6S-NBC20-VCR-C-CP  | Y   | 1   |
| SC09-6S-NBC20-VCR-C-NCP | Y   | 1   |
| SC10-6S-NBC20-VCR-E     | Y   | 1   |
| SC11-6S-NBC20-VCR-E-CP  | Y   | 1   |
| SC12-6S-NBC20-VCR-E-NCP | Y   | 1   |
| SC19-3S-NBC20-VCR-C     | Y   | 1   |
| SC20-3S-NBC20-VCR-C-CP  | Y   | 1   |
| SC21-3S-NBC20-VCR-C-NCP | Y   | 1   |
| SC22-3S-NBC20-VCR-E     | Y   | 1   |
| SC23-3S-NBC20-VCR-E-CP  | Y   | 1   |

Table 5.1 Considered scenarios (continue).

|                              |   |   |
|------------------------------|---|---|
| SC24-3S-NBC20-VCR-E-NCP      | Y | 1 |
| SC30-6S-NBC20-VCR-C-RdRo=3.9 | Y | 2 |
| SC31-3S-NBC20-VCR-C-RdRo=3.9 | Y | 2 |
| SC32-6S-NBC20-VCR-E-CP-A23   | Y | 2 |
| SC33-3S-NBC20-VCR-E-CP-A23   | Y | 2 |

The summary of main recorded response parameters is given in Table 5.2. When needed, additional response parameters were recorded to provide a better understanding of the system behaviour.

Table 5.2 Parameters measured.

| No. | Parameters                | Symbol    | Type           | Comments                                                             |
|-----|---------------------------|-----------|----------------|----------------------------------------------------------------------|
| 1   | Modal periods             | T         | Direct lecture | None                                                                 |
| 2   | Lateral displacement      | $D_x$     | Direct lecture | Taken at each storey                                                 |
| 3   | Base force                | $P_b$     | Direct lecture | None                                                                 |
| 4   | Column reaction           | $P_c$     | Direct lecture | Lecture taken at first Gauss -point above footing                    |
| 5   | Footing vertical movement | $D_z$     | Direct lecture | Lecture taken at each footing-spring node                            |
| 6   | Spring reaction           | $R_z$     | Direct lecture | Lecture taken at each spring-fixed node                              |
| 7   | Inter-storey drift        | $D_x$     | Processed      | Computed using lateral displacement (point 2) and storey height      |
| 8   | Overturning moment        | M         | Processed      | Computed as the sum of 4 and vertical component of 3 times lever arm |
| 9   | Base shear                | V         | Processed      | Computed as the sum the horizontal component of 3                    |
| 10  | Footing rotation          | $\theta$  | Processed      | Computed with the vertical variation of 5 along the footing length   |
| 11  | Moment rotation           | $M_{rot}$ | Processed      | Correlation of 8 and 10                                              |

## 5.2 Periods of vibration

The motivation to determine the fundamental period of vibration of the frame-foundation-soil system lies in the fact that SSI implementation normally decreases structural stiffness resulting in period elongation. Table 5.3 shows the fundamental periods for Vancouver scenarios for the Set 1 and Set 2 results. The comparison with fixed-base frame is shown through the ratio  $T_{SSI} / T_{FIX}$ .

Table 5.3 Modal period results for Vancouver with and without SS inclusion

| Scenario       | Foundation type | $T_{1, FIX}$ | $T_{1, SSI}$ | $T_{1, SSI} / T_{1, FIX}$ |
|----------------|-----------------|--------------|--------------|---------------------------|
| 6S-NBC20-VCR-C | CP              | 0.79         | 0.82         | 1.03                      |
|                | NCP             |              | 0.82         | 1.04                      |
|                | $R_d R_o = 3.9$ |              | 0.93         | 1.18                      |
| 3S-NBC20-VCR-C | CP              | 0.63         | 0.65         | 1.03                      |
|                | NCP             |              | 0.65         | 1.03                      |
|                | $R_d R_o = 3.9$ |              | 0.65         | 1.03                      |
| 6S-NBC20-VCR-E | CP              | 0.58         | 0.63         | 1.08                      |
|                | NCP             |              | 0.64         | 1.09                      |
|                | A23             |              | 0.58         | 1.00                      |
| 3S-NBC20-VCR-E | CP              | 0.36         | 0.39         | 1.09                      |
|                | NCP             |              | 0.40         | 1.10                      |
|                | A23             |              | 0.36         | 1.00                      |

Since the modal period are exclusively mass and stiffness related parameter (damping can be neglected for small critical damping ratios) and because the mass remains constant between scenarios, it is possible to conclude that a stiffness reduction is taking place as the period increase when fixed base models and SSI implemented models are compared.

For all but two scenarios, the elongation of the period occurs when the SSI is implemented. This observation confirms the premise that SSI implementation does have a global influence on the response of the structure. The results show that as the foundation decreases, the period elongation increases. Indeed, the most pronounced period elongation occurs for the case where no capacity design of foundation is implemented. On the other hand, if the foundation is large enough, the fixed base response is achieved even if SSI is implemented.

The stiffness of the soil beneath the foundation has an important influence by reducing the rigidity of the overall system. SSI impact on period elongation is more pronounced for softer sites in comparison to stiffer sites.

### 5.3 Lateral displacement

The lateral displacement of the building is measured by the lateral inter-storey drift index which is defined as the differences in displacement of two consecutive storeys divided by the inter-storey height. NBC limits the inter-storey drift to 2.5% for buildings other than post-disaster buildings and High Importance Category buildings. The following figure presents the drift index of the set 1 for three and six storeys buildings located in Vancouver for CP and NCP foundations as well as for those models with fixed base.

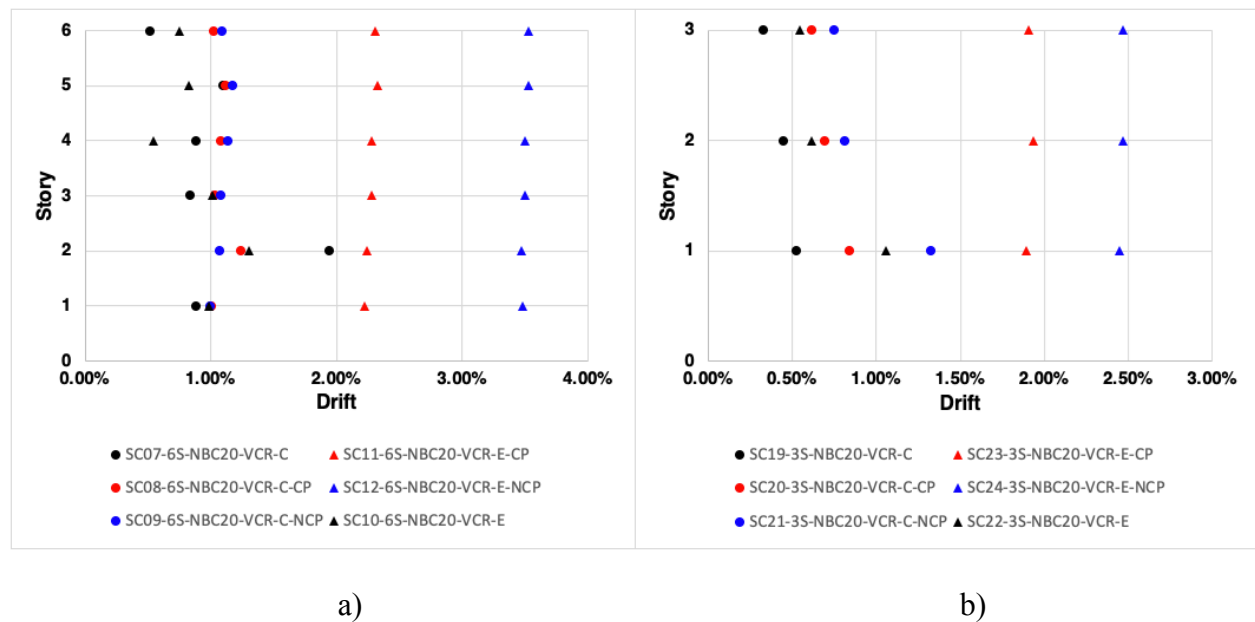


Figure 5.1 Interstorey drift index a) Vancouver six storey building, b) Vancouver three storey building.

Inn Figure 5.1, circles show results for Class C site, while triangles represents Class E site. The lateral drift index of the building on site Class E is much more prominently affected by the footing rotation when SSI is implemented in the model compared to the site C. Further examination of the shape of the plots, confirms that the superstructure behaves as a rigidity body which rotates due to the settlement and the uplift of the footing. This response can be seen for both frame heights. For CP foundations, in any case the drift index exceeds the permitted limit. In fact, for these foundations when site Class C is implemented, the drift index is well below 2.5%. For frames with

NCP foundations, the NBCC drift index limits exceeded for 6-storeys frame and it almost reached the limit in the case of three-storey frame.

In general, results for scenarios on site C, show less dispersion compared to site E. Drift indexes for NCP foundations are larger than those recorded for fixed-base and CP scenarios, except at the second story of the six-storey building. Closer examination of brace behaviour at this level shows that important brace buckling and yielding occur in this particular scenario causing a large inelastic displacement.

The major observation concerning the inter-storey drift index is the increase due to the SSI impact for the softer soil (site Class E). For the frames with fixed base, the overall drift response for site Class C and for site Class E is rather similar and stays well below the NBCC limit of 2.5%.

Table 5.4 inter-storey drift index for Vancouver scenarios.

| Scenario                | Storey |       |       |       |       |       | Max   |
|-------------------------|--------|-------|-------|-------|-------|-------|-------|
|                         | 1      | 2     | 3     | 4     | 5     | 6     |       |
| SC07-6S-NBC20-VCR-C     | 0.88%  | 1.94% | 0.83% | 0.88% | 1.10% | 0.52% | 1.94% |
| SC08-6S-NBC20-VCR-C-CP  | 1.00%  | 1.23% | 1.03% | 1.07% | 1.12% | 1.02% | 1.23% |
| SC09-6S-NBC20-VCR-C-NCP | 0.99%  | 1.07% | 1.07% | 1.14% | 1.17% | 1.09% | 1.17% |
| SC10-6S-NBC20-VCR-E     | 0.98%  | 1.30% | 1.01% | 0.55% | 0.83% | 0.75% | 1.30% |
| SC11-6S-NBC20-VCR-E-CP  | 2.22%  | 2.24% | 2.28% | 2.28% | 2.32% | 2.31% | 2.32% |
| SC12-6S-NBC20-VCR-E-NCP | 3.48%  | 3.47% | 3.50% | 3.50% | 3.53% | 3.52% | 3.53% |
| SC19-3S-NBC20-VCR-C     | 0.52%  | 0.44% | 0.33% | --    | --    | --    | 0.52% |
| SC20-3S-NBC20-VCR-C-CP  | 0.84%  | 0.69% | 0.62% | --    | --    | --    | 0.84% |
| SC21-3S-NBC20-VCR-C-NCP | 1.33%  | 0.81% | 0.75% | --    | --    | --    | 1.33% |
| SC22-3S-NBC20-VCR-E     | 1.06%  | 0.62% | 0.55% | --    | --    | --    | 1.06% |
| SC23-3S-NBC20-VCR-E-CP  | 1.89%  | 1.93% | 1.91% | --    | --    | --    | 1.93% |
| SC24-3S-NBC20-VCR-E-NCP | 2.45%  | 2.47% | 2.46% | --    | --    | --    | 2.47% |

## 5.4 Brace forces

Internal forces in the diagonals are another key result that can be helpful to describe the response of the frames. Since they are designed as primary ductile elements with the role to develop inelastic deformations through yielding and buckling, it is expected that the maximum forces developed in these elements will be limited by their tensile and compressive buckling capacity. Figures 5.2 and 5.3 illustrate the maximum internal diagonals forces normalized to  $T_{prob}$  for tensile forces and to  $C_{prob}$  for compressive forces.

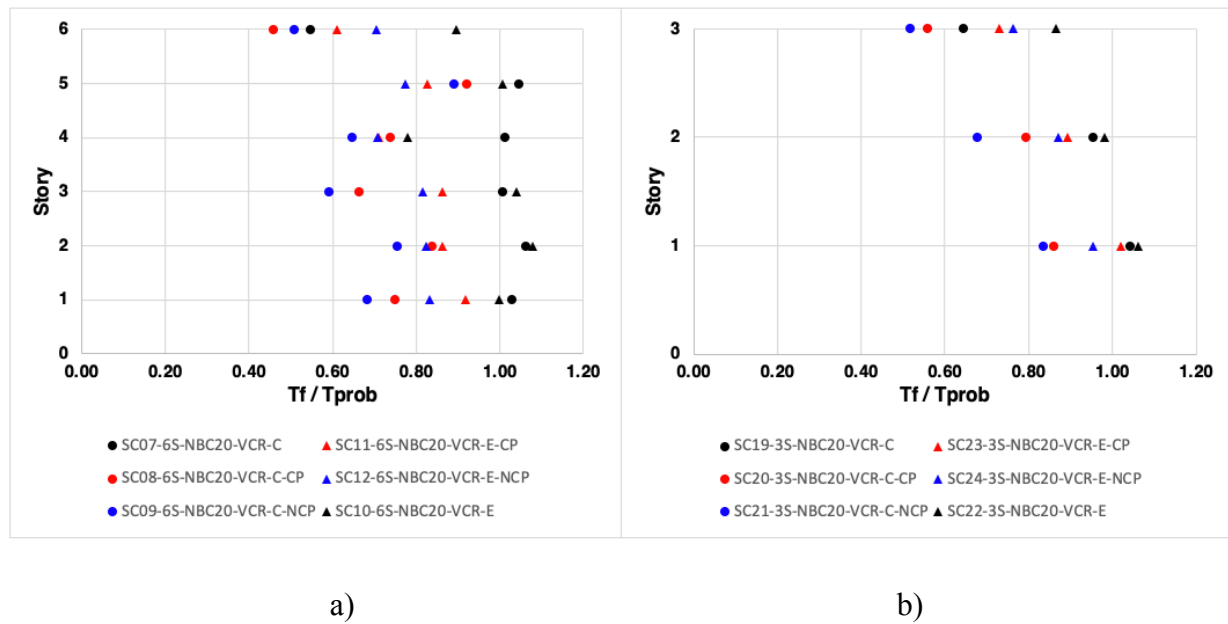


Figure 5.2 Internal forces in the diagonals normalized to  $T_{prob}$ . a) Six-storeys scenarios. b) Three-storey scenarios.

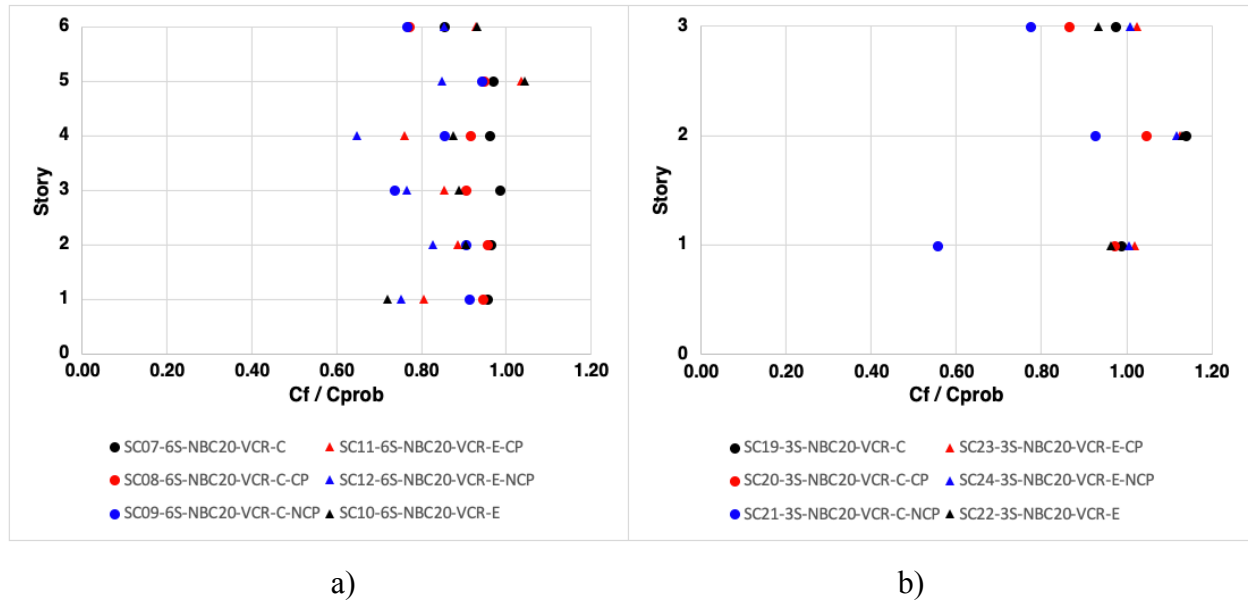


Figure 5.3 Internal forces in the diagonals normalized to  $C_{prob}$ . a) Six-storeys scenarios. b) Three-storey scenarios.

Figure 5.2 shows that fixed-base scenarios are those that result in the highest tensile forces in the diagonals. For six-storey frames, with exception of the top storey, all braces yield in tension. Similarly, for the three-storey building, the last story remains elastic. When the SSI is included in the analysis, non-linear response of soil-foundation system limits significantly seismic input into the superstructure. Generally, the smaller the foundation, the greater decrease in the brace maximal tensile forces. Response in compression show a similar trend, however, in this case the difference between the upper storey and the all others is less accentuated.

It should be noticed that for some cases, specially at the second storey in Figure 5.3 (b), the internal forces in the braces surpass the probable capacity of the elements. This can be explained by influence of the gusset plate that has been considered in the modeling which causes the differences in the buckling length considered in the design and in the analysis. For the tensile forces, the occurrence of the strain hardening can explain the phenomena.

An interesting way to inspect the influence of the SSI and the foundation size on the responses of the braces is by taking a look into their hysteretic curves. Figure 5.4 shows the hysteretic response of a selected brace for a fixed-base as well as CP and NCP foundation. For the fixed-base case, the curve corresponds to the typical brace hysteretic response reported from experimental testing under cycling loading. The behaviour is initially linear-elastic until the probable tensile capacity is reached. Once yielding occurs, a plastic plateau forms. Inelastic displacements increase considerably with no substantial force increase. For compression loading, the ultimate compressive capacity is reached in only one cycle before the brace buckles. When SSI is added, the shape of the hysteresis changes completely to a more rounded form due to the rotational movement of the footing. When SSI is implemented, the foundation movements and the additional damping added into the system limit the input of seismic energy to the superstructure and no yielding due to tensile forces nor buckling due to compressive force occur in this specific case.

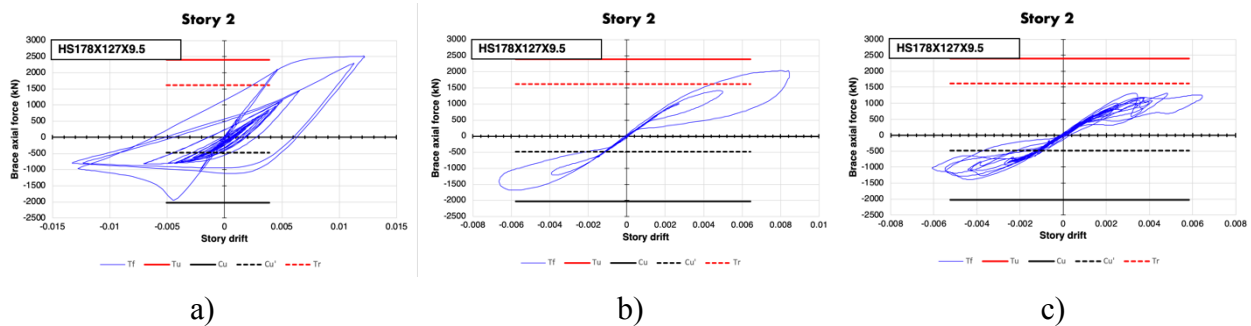


Figure 5.4 Hysteresis curves for second storey brace for six storey building located in Vancouver on site C. a) Fixed base, b) CP foundation, c) NCP foundation.

## 5.5 Overturning moment

Since the overturning moment describes the rotational movement of the foundations and set the force level to which foundations are designed, the overturning moments developed in the numerical simulations,  $M_f$ , were closely examined. To determine the overturning moment that the braced frame transfers to the foundation, the axial force at the base and the column and the vertical component of the first story braces were added and multiplied by the appropriate lever arms at every time step and for each ground motion. The calculated moment is then correlated to the rotation of the footing at every step of the analyses. In this way, a moment is related to a corresponding rotation. By tracking this correlation during the complete length of the analyses, (i.e. at each time step), it was possible to identify the level of demand that triggers the inelastic rotation of the footing; the moment when one of the foundation edges reaches the zero-stress condition and the uplift begins. This overturning moment magnitude was denoted as  $M_{rot}$ . In order to discuss and compare  $M_f$  and  $M_{rot}$ , two calculations were done. In Table 5.5, results were normalized with respect to the probable overturning moment. This provides for a direct comparison with the maximum moment that the frame is able to sustain and with the foundation design moment. In Table 5.6, an equivalent  $R_d R_o$  were calculated needed to reduce the seismic elastic demand to the moment developed by the time-history analyses. This helps to compare the results with respect to the design moment.

It should be notice that two probable moment are presented in the results.  $M_{pr}$ , describes the actual probable moment of the modelled frame.  $M_{pr,design}$ , indicates the probable moment assumed in design of the frames and the foundations. Slightly differences exist between the two because of the length of the rigid offset due to the connections dimensions that is introduced in the numerical model. In general, shorter braces will have a higher probable capacity. Ratios between these two results ( $M_{pr}/M_{pr,design}$ ) are given in the Table 5.6.

Table 5.5 Normalized developed overturning moment.

| Scenario                | Normalized moment to $M_{pr}$ |                |                |                |                    |
|-------------------------|-------------------------------|----------------|----------------|----------------|--------------------|
|                         | $M_{pr} / M_{pr,design}$      | $M_f / M_{pr}$ | $M_d / M_{pr}$ | $M_n / M_{pr}$ | $M_{rot} / M_{pr}$ |
| SC07-6S-NBC20-VCR-C     | 1.17                          | 0.90           | --             | 0.65           | --                 |
| SC08-6S-NBC20-VCR-C-CP  | 1.17                          | 0.82           | 0.85           | 0.65           | 0.36               |
| SC09-6S-NBC20-VCR-C-NCP | 1.17                          | 0.69           | 0.72           | 0.65           | 0.36               |
| SC10-6S-NBC20-VCR-E     | 1.20                          | 0.88           | --             | 0.62           | --                 |
| SC11-6S-NBC20-VCR-E-CP  | 1.20                          | 0.67           | 0.83           | 0.62           | 0.29               |
| SC12-6S-NBC20-VCR-E-NCP | 1.20                          | 0.70           | 0.72           | 0.62           | 0.29               |
| SC19-3S-NBC20-VCR-C     | 1.14                          | 1.00           | --             | 0.67           | --                 |
| SC20-3S-NBC20-VCR-C-CP  | 1.14                          | 0.99           | 0.88           | 0.67           | 0.52               |
| SC21-3S-NBC20-VCR-C-NCP | 1.14                          | 0.82           | 0.78           | 0.67           | 0.52               |
| SC22-3S-NBC20-VCR-E     | 1.14                          | 1.00           | --             | 0.65           | --                 |
| SC23-3S-NBC20-VCR-E-CP  | 1.14                          | 0.98           | 0.88           | 0.65           | 0.49               |
| SC24-3S-NBC20-VCR-E-NCP | 1.14                          | 0.94           | 0.73           | 0.65           | 0.49               |

Table 5.6 Equivalent  $R_d R_o$  for overturning moments.

| Scenario                | Equivalent $R_d R_o$ |       |       |       |           |
|-------------------------|----------------------|-------|-------|-------|-----------|
|                         | $M_{pr}$             | $M_f$ | $M_d$ | $M_n$ | $M_{rot}$ |
| SC07-6S-NBC20-VCR-C     | 1.43                 | 1.59  | --    | 2.21  | --        |
| SC08-6S-NBC20-VCR-C-CP  | 1.43                 | 1.75  | 1.68  | 2.21  | 4.00      |
| SC09-6S-NBC20-VCR-C-NCP | 1.43                 | 2.07  | 2.00  | 2.21  | 4.00      |
| SC10-6S-NBC20-VCR-E     | 1.45                 | 1.66  | --    | 2.33  | --        |
| SC11-6S-NBC20-VCR-E-CP  | 1.45                 | 2.15  | 1.74  | 2.33  | 5.00      |
| SC12-6S-NBC20-VCR-E-NCP | 1.45                 | 2.08  | 2.00  | 2.33  | 5.00      |
| SC19-3S-NBC20-VCR-C     | 1.55                 | 1.55  | --    | 2.30  | --        |
| SC20-3S-NBC20-VCR-C-CP  | 1.55                 | 1.56  | 1.77  | 2.30  | 3.00      |
| SC21-3S-NBC20-VCR-C-NCP | 1.55                 | 1.88  | 2.00  | 2.30  | 3.00      |
| SC22-3S-NBC20-VCR-E     | 1.47                 | 1.47  | --    | 2.24  | --        |
| SC23-3S-NBC20-VCR-E-CP  | 1.47                 | 1.50  | 1.67  | 2.24  | 3.00      |
| SC24-3S-NBC20-VCR-E-NCP | 1.47                 | 1.56  | 2.00  | 2.24  | 3.00      |

Table 5.5 shows that stiffer the base of the frame, a bigger moment is developed. Indeed, for fixed base models of three-storey frames, the developed moment goes as high as the probable moment, which is consistent with the design assumption that inelastic braces response limits the seismic input into the superstructure. For six-storey frames, the probable moment is not reached, even though, for fixed base,  $M_f$  reaches as high as 90% of  $M_{pr}$ .

Consistency with design provisions can also be observed for the overturning moment that develops for frames with CP foundation for the scenarios SC20 and SC23 for which  $M_f$  reaches  $M_{pr}$ . For scenarios SC08 and SC11, the influence of the SSI on structural response is significant as the frames attain only 82% and 67% of  $M_{pr}$  respectively. For NCP scenarios, the influence of SSI on the results can be identified, being more important for the six-storey frames with a developed moment of  $0.7 M_{pr}$ .

Due to the differences between  $M_{pr}$  and  $M_{pr,design}$  exposed earlier, the moment used in the CP design does not match the code estimates. For NCP scenarios the differences between  $M_{pr}$  and  $M_{pr,design}$  do not have a tangible impact in the footing dimensions. The moment used for the design of the foundation ( $M_d$ ), corresponds to  $M_{pr,design}$  for CP and the maximum between 75% of the nominal moment of CBF and the moment equivalent to  $R_d R_o = 2$ . It can be seen in Table 5.5 that the  $M_d/M_{pr}$  ratio is actually less than 1 because of the differences between  $M_{pr}$  and  $M_{pr,design}$ . From Table 5.6 it can be observed that the  $M_{R_d R_o = 2}$  governs the design for NCP.

The comparison is also made with respect to the nominal moment,  $M_n$ . A consistent value of  $M_n$  normalized with  $M_{pr}$  of around 0.65 is observed for all scenarios. Thus, the nominal strength of the CBF is equivalent to the 65% of its probable capacity. Or in other words, the probable capacity of the CBF is about 1.54 times the nominal resistance of the frame ( $1/0.65=1.54$ ).

Also, two trends are identified for the rotation-trigger moment. For different six-storeys frame scenarios, an overturning moment of around  $0.30 M_{pr}$  (between  $R_d R_o = 4$  and  $R_d R_o = 5$ ) is sufficient to start a significant uplift and rotation of the foundation while for three-storeys frame a bigger overturning moment of about  $0.5 M_{pr}$  ( $R_d R_o = 3$ ) is required.

Finally, no significant differences between site C and site E scenarios were identified for the overturning moment results. Slightly differences exist, however the reason for this variability in the results is related to the type of foundation, the boundary conditions at the base (fixed or not) and in the height of the frame.

## 5.6 Base shear

As the overturning moment serves to describe the rotation movement of the foundations and the capacity of both frame and foundation, the base shear developed by the CBF during time-history analyses helps to quantify the input of seismic force into the system by comparing the base shear recorded for the fixed base models with those where SSI were considered. Table 5.7 shows the base shear related results. The ratio  $V_f/W$  is known as *seismic coefficient* and it expresses the equivalent lateral force as portion of the seismic weight,  $W$ . The *seismic input reduction coefficient* is the ratio between seismic coefficient in a given scenario and the equivalent fixed-base scenario. In other words, it expresses by how much the SSI is able to reduce the lateral demands on the superstructure.

Another base shear related parameter worthy examining is the sliding. As discussed in the first chapter, no lateral springs were employed to model possible sliding failure. Nevertheless, in order to permit a more general interpretation of results, this failure mode was examined in a simplified manner by comparing the base shear developed by the model to a base shear needed to surpass the friction resistance of the footing-soil interface.

Base shear results from time-history analyses are presented in Table 5.7.

Table 5.7 Base shear results.

| Scenario                | $V_f$ | $V_{Sliding}$ | $V_f/V_{Sliding}$ | Sliding may occur | $V_f/W$ | Seismic input reduction coefficient |
|-------------------------|-------|---------------|-------------------|-------------------|---------|-------------------------------------|
| (--)                    | kN    | kN            | (--)              | (--)              | (--)    | (--)                                |
| SC07-6S-NBC20-VCR-C     | 3138  | --            | --                | --                | 0.29    | --                                  |
| SC08-6S-NBC20-VCR-C-CP  | 3055  | 4149          | 0.74              | N                 | 0.28    | 0.97                                |
| SC09-6S-NBC20-VCR-C-NCP | 2930  | 3932          | 0.75              | N                 | 0.27    | 0.93                                |
| SC10-6S-NBC20-VCR-E     | 5020  | --            | --                | --                | 0.46    | --                                  |
| SC11-6S-NBC20-VCR-E-CP  | 4590  | 8862          | 0.52              | N                 | 0.42    | 0.91                                |
| SC12-6S-NBC20-VCR-E-NCP | 4809  | 8253          | 0.58              | N                 | 0.44    | 0.96                                |
| SC19-3S-NBC20-VCR-C     | 2613  | --            | --                | --                | 0.51    | --                                  |
| SC20-3S-NBC20-VCR-C-CP  | 2382  | 2077          | 1.15              | Y                 | 0.47    | 0.91                                |
| SC21-3S-NBC20-VCR-C-NCP | 2031  | 1966          | 1.03              | Y                 | 0.40    | 0.78                                |
| SC22-3S-NBC20-VCR-E     | 3886  | --            | --                | --                | 0.77    | --                                  |
| SC23-3S-NBC20-VCR-E-CP  | 4495  | 3365          | 1.34              | Y                 | 0.88    | 1.16                                |
| SC24-3S-NBC20-VCR-E-NCP | 3835  | 3158          | 1.21              | Y                 | 0.76    | 0.99                                |

The results given in Table 5,7 are interesting and consistent with other results. With the exception of scenarios SC23 and SC24, SSI actually reduces the lateral demands on the structure, and, as other results have already shown up to this point, the smaller the foundation, the greater the influence of the SSI on the overall response. Scenarios SC23 and SC24 (three-storey frames located on site Class E site), show that for the stiffer structure on the softer soil, SSI may be unbeneficial. The sliding of the frame has no major impact in most cases.  $V_f/V_{sliding}$  ratio is greater than 1 for scenarios SC20 and SC21, but only slightly. Scenarios SC23 and SC24, which corresponds to the stiffer framer configuration, may have an important effect due to sliding, therefore a closer inspection was made to identify during the analyses the other parameters such as footing rotation, settlements, uplift and overturning moment and thus assess the impact of possible foundation sliding on results. It was found that non-negligible rotation may occur before the base shear reaches the  $V_{sliding}$ , however, maximum values of rotation, uplift and settlements were recorded after. For these scenarios a more detailed model which incorporates horizontal springs may be necessary to capture properly the impact of sliding on overall response.

## 5.7 Soil bearing demands

As discussed in the literature review, the procedure followed for the design of the foundation serves to avoid soil yielding and to ensure equilibrium (prevent overturning of the frame). One parameter intended to examine the response and the performance of the soil beneath the foundation is the soil bearing demands along the foundation. To do so, the lecture of the vertical reaction is taken and they are normalized to the ultimate capacity of the spring,  $q_{ult}$ . This last one is estimated by the ultimate capacity of the soil assumed in the design and multiplied it by the tributary area of the spring. Next figures show the soil bearing demand normalized to the ultimate capacity of the soil.

As discussed in the Chapter 2, the procedure for seismic design of foundations aims to avoid soil yielding and to ensure equilibrium i.e. prevent an overturning of the frame. To estimate soil bearing pressure along the foundation length, vertical reaction in the springs were recorded and normalized by the ultimate capacity of the spring,  $q_{ult}$ . The latter is determined by multiplying the ultimate capacity of the soil assumed in the design by the tributary area of the spring. Figures 5.5 to 5.8 show normalized soil bearing along the length of the foundation on site Class C. These curves are envelopes and represent the maximum normalized compressive for at each spring. Because soil completely yields along the length of the foundation when site Class E where considered, similar curves for such cases were not presented as they have an uniform value of 1 all along the length of the footing.

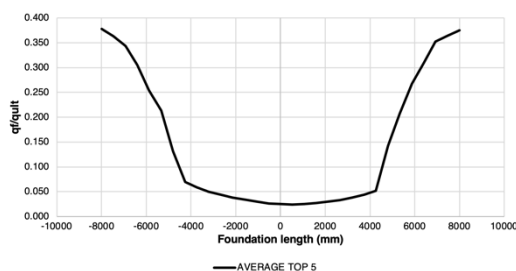


Figure 5.6 Normalized soil pressure  
SC09-6S-NBC20-VCR-C-NCP

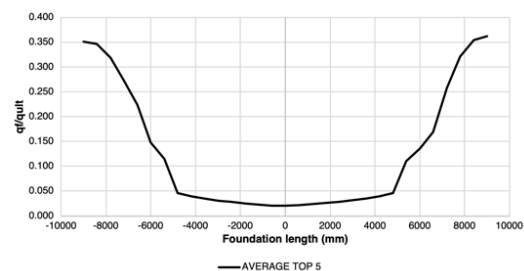


Figure 5.5 Normalized soil pressure  
SC08-6S-NBC20-VCR-C-CP

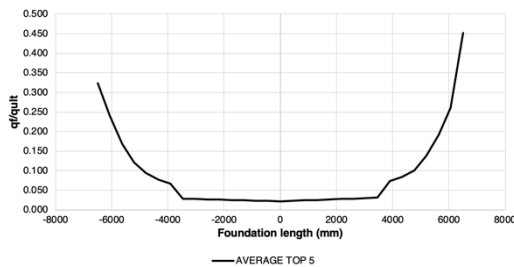


Figure 5.8 Normalized soil pressure  
SC21-3S-NBC20-VCR-C-NCP

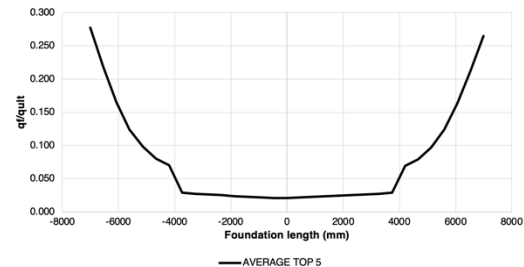


Figure 5.7 Normalized soil pressure  
SC20-3S-NBC20-VCR-C-CP

As anticipated, the maximal spring forces arise under foundation edges and then significantly decrease under the central portion of the foundation. They are slightly higher for three-storey frames, reaching the maximum value of 0.45 but in no case do they exceed the factored bearing resistance of the soil equal to  $0.5 q_{ul}$ . These results are consistent with previous work reported by Murugananathan (2018). NCP foundations present a slightly higher demands on the soil compared with CP, which is understandable since the dimensions of NCP foundations are smaller.

Results obtained for Class E site are considerably different. In fact, yielding occurs all along the length of the foundation which causes a substantial permanent settlement that will be discussed later. It seems that the rocking response of footing introduce very high forces in the springs reaching to the yielding level. The analysis of the soil yielding sequence shows that the yielding start at the footing tips, as anticipated. A chain reaction mechanism is then triggered. With the first spring yielded and no increment in the restoring force in that spring, the force on the adjacent spring increases leading eventually to plastification. The similar sequence affects eventually all springs and after several cycles of earthquake loading, all the springs appear to end up in post-yielding state. This response was judged not acceptable for a building and raised questions of convergence so further inspection was made. Discussion of the findings is presented in section 5.11.

## 5.8 Settlements and uplift

Recorded vertical movement of foundations is denoted as settlement for downward movement and uplift for upward movement. This parameter was determined at each foundation node where SSI springs are connected. Figures 5.9 to 5.12 and 5.13 to 5.16 illustrate results obtained for Vancouver scenarios, for Class C and Class E sites, respectively.

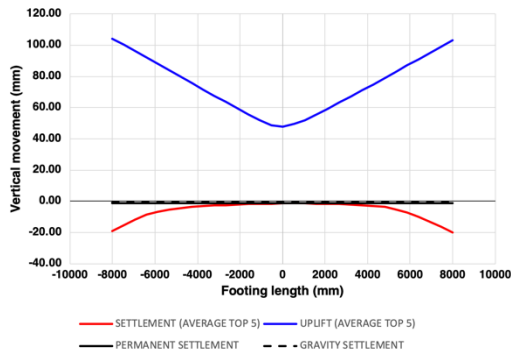


Figure 5.10 Footing vertical displacements for scenario SC09-6S-NBC20-VCR-C-NCP

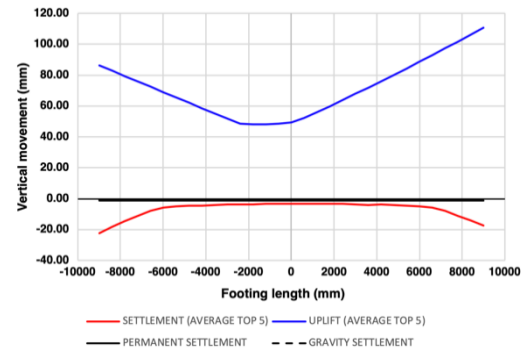


Figure 5.9 Footing vertical displacements for scenario SC08-6S-NBC20-VCR-C-CP

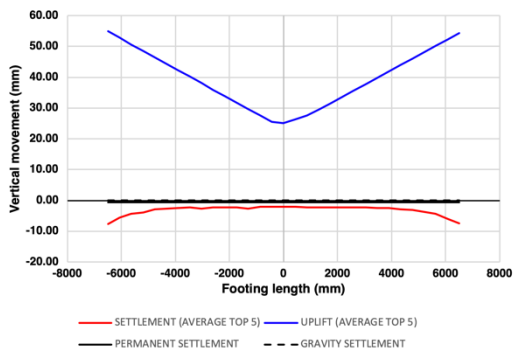


Figure 5.12 Footing vertical displacements for scenario SC21-3S-NBC20-VCR-C-NCP

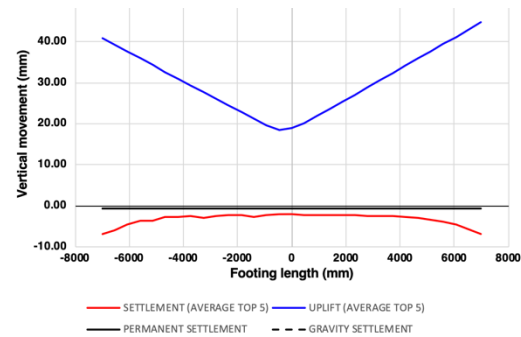


Figure 5.11 Footing vertical displacements for scenario SC20-3S-NBC20-VCR-C-CP

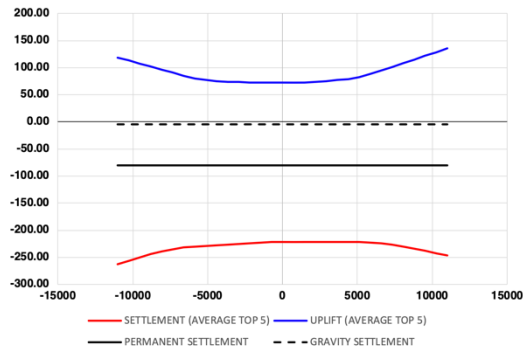


Figure 5.14 Footing vertical displacements for scenario SC12-6S-NBC20-VCR-E-NCP

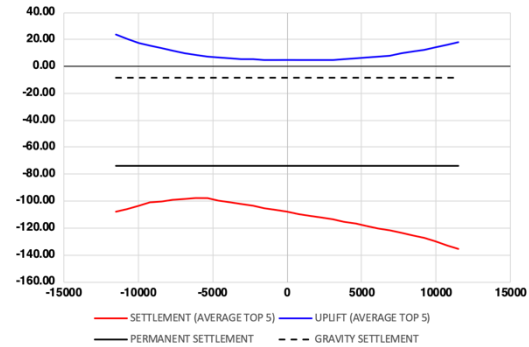


Figure 5.13 Footing vertical displacements for scenario SC11-6S-NBC20-VCR-E-CP

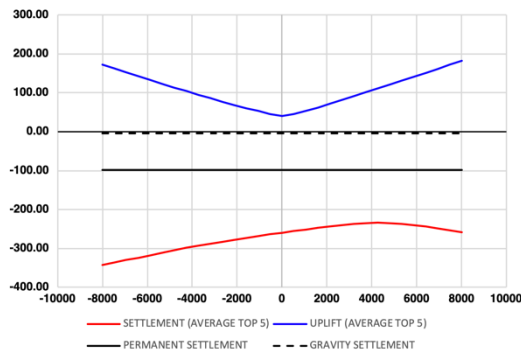


Figure 5.16 Footing vertical displacements for scenario SC24-3S-NBC20-VCR-E-NCP

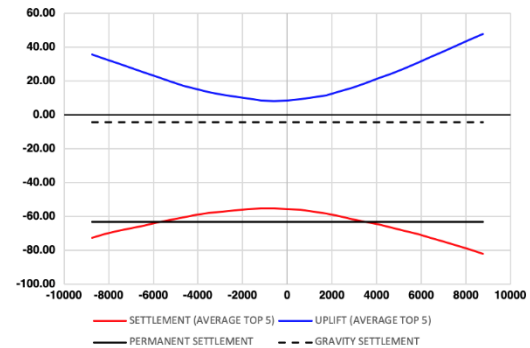


Figure 5.15 Footing vertical displacements for scenario SC23-3S-NBC20-VCR-E-CP

Similarly, to the result for soil bearing pressure demands, considerable differences between the response on site C and E can be seen. For Class C site, rocking movement occurs over a stiff surface. The maximum uplift ranges between 55 mm and 100 mm for three- and six-storey scenarios, respectively, while the settlement is confined to the ends of the footing with values varying between 10 mm and 20 mm, respectively. These values are judged acceptable since the maximum settlement is lower than the recommended limit of 25 mm and the permanent settlement at the end of the analysis is negligible (less than 2 mm).

Conversely, the soft soil of class E site lead to quite different observations. A larger settlement is noticeable along the footing length, not only at the ends. The rocking does occur but it is accompanied with the enforcement of the foundation into the soil on the opposite edge of the foundation causing important settlement. An asymmetrical response is also observed, which lead to a permanent lateral displacement, leaving the frame tilted at the end of the earthquake. In addition, permanent settlements exceeded a 25 mm limit, reaching a maximum of 330 mm.

## 5.9 Footing rotation

The rotation of the foundation can be estimated by the differences in the displacements at both ends of the footing and the length of the foundation. The rotation calculated in this manner were then compared with design estimates obtained by the empirical procedures specified in A23.3-19. Table 5.8 show the rotation design estimates ( $R_{A23.3}$ ), and the rotation obtained from time-history analysis ( $R_{OPENSEES}$ ).

Table 5.8 Footing rotation estimates according to A23.3-13 simplified procedure and rotations from the nonlinear time-history analysis with SSI implemented.

| Scenario                | $R_{A23.3}$ | $R_{OPENSEES}$ | $R_{OPENSEES} / R_{A23.3}$ |
|-------------------------|-------------|----------------|----------------------------|
| SC07-6S-NBC20-VCR-C     | --          | --             | --                         |
| SC08-6S-NBC20-VCR-C-CP  | 0.0736      | 0.0152         | 0.2069                     |
| SC09-6S-NBC20-VCR-C-NCP | 0.0687      | 0.0170         | 0.2473                     |
| SC10-6S-NBC20-VCR-E     | --          | --             | --                         |
| SC11-6S-NBC20-VCR-E-CP  | 0.0207      | 0.0172         | 0.8349                     |
| SC12-6S-NBC20-VCR-E-NCP | 0.0205      | 0.0200         | 0.9779                     |
| SC19-3S-NBC20-VCR-C     | --          | --             | --                         |
| SC20-3S-NBC20-VCR-C-CP  | 0.0603      | 0.0075         | 0.1246                     |
| SC21-3S-NBC20-VCR-C-NCP | 0.0724      | 0.0101         | 0.1397                     |
| SC22-3S-NBC20-VCR-E     | --          | --             | --                         |
| SC23-3S-NBC20-VCR-E-CP  | 0.0254      | 0.0337         | 1.3270                     |
| SC24-3S-NBC20-VCR-E-NCP | 0.0228      | 0.0402         | 1.7676                     |

Foundation rotation results show similar trends as those for spring forces and vertical displacements. For three- and six-storey frames on Class C site  $R_{OPENSEES}$ , is a much smaller than A23.3-19 estimate, reaching the maximum value of 0.0170 rad and 0.0736, respectively. These results suggest, that in design process it would not require to increase the footing dimensions stipulated by A23.3-19 to minimize the additional inter-storey drift index as explained previously. So for this case, neglecting drift requirement related to footing rotations seems appropriate for design.

The same reasoning could not be applied for the frames on Class E site, for which the foundations develop a much larger rotation with the maximum reaching a  $R_{OPENSEES}$ -to-  $R_{A23.3}$  ratio up to 1.77, meaning that a greater rotation than estimates occurs.

### 5.10 Opportunities on site C

After the inspection of the different parameters recorded during the nonlinear time-history analyses for site Class C structures, several conclusions can be formulated. The response for all scenarios reveals that an appropriate structural response can be obtained with smaller foundations dimensions than those presently required by design provisions. As previously explained, compliance with the increased inter-storey drift requirements resulting from foundation rotations would have yielded foundation dimensions up to 1.37 times greater than the ones used in the analysis. To further optimize foundation dimensions, a set of additional analyses were made to determine the minimum foundation size that still provide a satisfactory response of superstructure-foundation-soil system. To do so, foundations were design for an overturning moment taken directly from the modal response spectrum analysis, with no capacity-related foundation overstrength. This corresponds to the equivalent overturning moment calculated with  $R_d R_o = 3.9$ . This approach is similar to the one employed by the American design provisions where is permitted to ignore any capacity design hierarchy to estimate foundations dimensions.

Foundations dimensions obtained for two new scenarios (three- and six-storey frames, Class C site, design overturning moment related to  $R_d R_o = 3.9$ ) are given in Table 5.9.

Nonlinear time-history analyses revealed a satisfactory structural response for both studied cases. Nevertheless, many parameters indicate that the performance of the soil-foundation-frame system was pushed up to the limits of the earthquake design objectives (life safety). A description and discussion of the results parameters is given below.

#### *Inter-storey drifts*

Figure 5.17 shows that for both cases studied, the inter-storey drifts are almost constant along the height of the frame which indicates that the frame rotates as a rigid body due to the SSI. For three-storeys scenario, drift index remained well below from the limit of 2.5% with maximum values reaching around 1.5% while for the six-storey scenario the drift index reached the design limit.

### *Diagonal demands*

Figure 5.18 shows that both compressive and tensile demands in braces are notably reduced. For both scenarios maximum compressive forces recorded range between 0.40 and 0.60 of the probable compressive capacity, while tensile demands range between 0.30 to 0.60 of the probable tensile capacity. No yielding nor buckling are expected. Footing rocking serves as an energy dissipation mechanism that limits effectively the input of seismic energy into the superstructure.

### *Overturning moment*

From Table 5.10 and Table 5.11 it can be seen that overturning moments are reduced significantly for both scenarios. with a larger reduction recorded for the six-storey frames ( $0.43 M_{pr}$  vs.  $0.61 M_{pr}$  for the three-storey frames). As expected, stiffer system, developed more force compared to the more flexible system. Additionally, because the foundation dimensions are smaller compared to CP and NCP designs, the SSI has the biggest impact on the results. The intense dissipation of seismic energy through foundation rocking, decreases significantly a demand on the superstructure which is protected and does not develop its probable capacity.

### *Base shear*

Table 5.12 confirms that the base shear is reduced compared with the base fixed model by a greater margin compared to CP and NCP foundations. The *seismic input reduction coefficients* were 0.61 and 0.57 for three- and six-storey frames, respectively, which are a remarkable reduction compared to the one observed for NCP foundation for which the base shear was reduced in comparison to fixed-base model by 0.78 and 0.93 for three- and six-storey frames respectively.

### *Soil bearing demands*

Soil bearing demands are large enough to induce yielding to occur along the springs place at the end regions. However, contrary to the site E results exposed previously, at the middle region of the foundation a demand less than  $0.20 q_{ult}$  was observed.

### *Settlements and uplift*

As seen from Table 5.13, the settlement of the soil under the foundations is augmented compared to CP and NCP cases because a larger portion of the soil underneath foundation yields. Up to 20 mm and 85 mm displacements were recorded for the 3- and 6- storeys respectively. Nevertheless, these maximum values are during the duration of the earthquake. At the end of the analysis, no permanent net settlement (settlement estimated without the influence of gravitational loads) was

recorded for 3-storey frame, while 12 mm was recorded for the six-storey frame. These values seem acceptable to avoid non-structural and structural damage. However, damage can be also expected during the earthquake for the taller frame because the large absolute settlement of 85 mm.

### *Footing rotation*

Table 5.14 summarizes results obtained for foundation rotations. obtained from the non-linear analyses for six stories frames is 0.0475 rad, which is a similar value of 0.0443 rad calculated from the A23.3 provisions. For the three-storey frame, maximum rotation is about half of design estimate (0.0227 rad vs. 0.0423 rad), while for the six-storey frame, design estimate seems appropriate (0.0475 rad vs 0.0443 rad).

Table 5.9 Foundation dimensions for scenarios SC30 and SC31.

| Scenarios                    | Length | Width | Thickness |
|------------------------------|--------|-------|-----------|
|                              | m      | m     | m         |
| SC30-6S-NBC20-VCR-C-RdRo=3.9 | 11     | 3     | 2         |
| SC31-3S-NBC20-VCR-C-RdRo=3.9 | 10     | 2.5   | 1.5       |

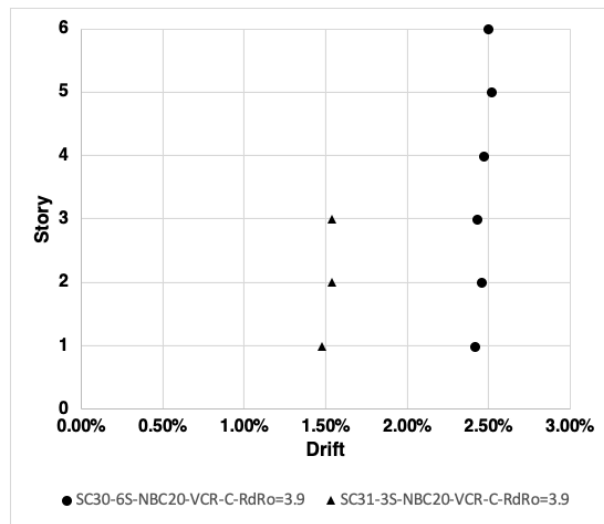


Figure 5.17 Inter storey drift for scenarios S30 and SC31.

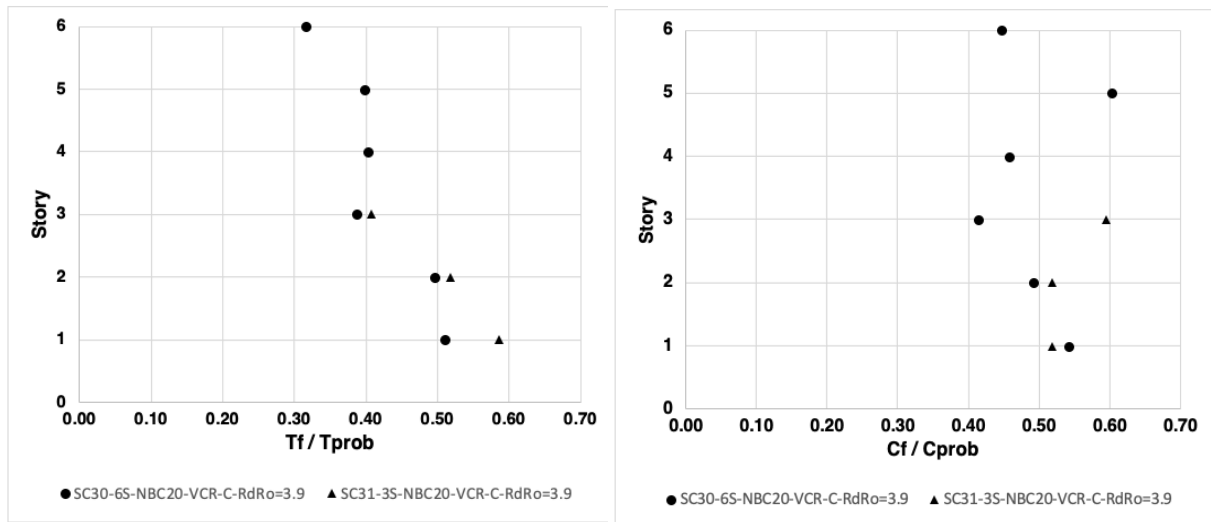


Figure 5.18 Diagonals demands for scenarios SC30 and SC31.

Table 5.10 Normalized developed overturning moment for scenarios SC30 and SC31.

| Scenario                     | Normalized overturning moment to $M_{pr}$ |                |                |                |                    |
|------------------------------|-------------------------------------------|----------------|----------------|----------------|--------------------|
|                              | $M_{pr} / M_{pr,desing}$                  | $M_f / M_{pr}$ | $M_r / M_{pr}$ | $M_n / M_{pr}$ | $M_{rot} / M_{pr}$ |
| SC30-6S-NBC20-VCR-C-RdRo=3.9 | 1.17                                      | 0.43           | 0.37           | 0.65           | 0.36               |
| SC31-3S-NBC20-VCR-C-RdRo=3.9 | 1.14                                      | 0.61           | 0.40           | 0.67           | 0.39               |

Table 5.11 Equivalent  $R_d R_o$  for overturning moment for scenarios SC30 and SC31.

| Scenario                     | Equivalent $R_d R_o$ |       |       |       |           |
|------------------------------|----------------------|-------|-------|-------|-----------|
|                              | $M_{pr}$             | $M_f$ | $M_r$ | $M_n$ | $M_{rot}$ |
| SC30-6S-NBC20-VCR-C-RdRo=3.9 | 1.43                 | 3.32  | 3.90  | 2.21  | 4.00      |
| SC31-3S-NBC20-VCR-C-RdRo=3.9 | 1.55                 | 2.55  | 3.90  | 2.30  | 4.00      |

Table 5.12 Base shear results for scenarios SC30 and SC31.

| Scenario                     | $V_f$<br>(kN) | $V_f/W$ | Seismic<br>input<br>reduction<br>coefficient |
|------------------------------|---------------|---------|----------------------------------------------|
| SC30-6S-NBC20-VCR-C-RdRo=3.9 | 1792          | 0.17    | 0.57                                         |
| SC31-3S-NBC20-VCR-C-RdRo=3.9 | 1606          | 0.32    | 0.61                                         |

Table 5.13 Footing settlements and uplift summary for scenarios SC30 and SC31.

| Scenario                     | Settlement <sub>min</sub><br>(mm) | Uplift <sub>max</sub><br>(mm) | Permanent<br>(mm) | Permanent net<br>(mm) |
|------------------------------|-----------------------------------|-------------------------------|-------------------|-----------------------|
| SC30-6S-NBC20-VCR-C-RdRo=3.9 | -85                               | 28                            | -13               | -12                   |
| SC31-3S-NBC20-VCR-C-RdRo=3.9 | -20                               | 23                            | -1                | 0                     |

Table 5.14 Footing estimated rotation by A23.3 provisions and by non-linear analysis with SSI implemented for scenarios SC30 and SC31.

| Scenario                     | $R_{A23.3}$ | $R_{OPENSEES}$ | $R_{OPENSEES} / R_{A23.3}$ |
|------------------------------|-------------|----------------|----------------------------|
| SC30-6S-NBC20-VCR-C-RdRo=3.9 | 0.0443      | 0.0475         | 1.0712                     |
| SC31-3S-NBC20-VCR-C-RdRo=3.9 | 0.0423      | 0.0227         | 0.5365                     |

### 5.11 In search forward of an appropriate response on Class E site

Close examination of the results obtained for Class E site revealed that the smaller soil capacity and smaller stiffens of the soil led to an inappropriate response of the system soil-foundation-structure. Large lateral displacements, failure of the soil and unacceptable settlements were observed. This motivated additional analyses to establish the necessary conditions for a satisfactory response. The reader is reminded that initial foundation design was not fully compliant with current requirements for foundation design as foundation dimensions were not increased to limit inter-storey drifts augmented by the rotation of foundations. New scenarios for site E were created with a capacity protected foundation dimensioned to meet all rotation-related requirements. This resulted 1.30 times larger footing dimensions in terms of surface area and they are showed in Table 5.15. This approach resulted in improved response that can be judged acceptable for the 3-storey building. For 6-storey building, most of the response parameters were acceptable nevertheless, significant settlements remained. This issue should be investigated further in the future studies. Details regarding the selected response parameters are given below.

#### *Inter-storey drifts*

As seen in Figure 5.19, inter-storey drift index of the superstructure was below the imposed limit for all storeys for both building heights. A constant inter-story drift is observed, which implies a rigid-body rotational response of the structure. A permanent drift index less that the limit due to a residual footing rotation was also observed.

#### *Brace demands*

In Figure 5.20 tensile yielding was only reached at the fist story of the three-stories building. Lateral bucking due to compressive demands was observed at all stories of the same scenario. All braces of the six-storey frame remained elastic.

#### *Overturning moment*

Results obtained for overturning moments, expressed in terms of overturning moments normalized by  $M_{pr}$  and equivalent  $R_d R_o$  factors are given in Tables 5.16 and 5.17, respectively. For the three-storey frame, only 76% of the probable moment of the frame was developed while the foundation was designed to sustain 109% of  $M_{pr}$  due to the rotation-related provisions. For 6-storey frame on the other hand developed moment that match between design overturning moment and time-history results was observed; 105% of  $M_{pr}$  developed for this frame.

### *Base shear*

As explained before, as the dimensions of the footings increase, the response approaches that of the fixed-base system, with limited impact of SSI. That behaviour was observed for both frame heights. As Table 5.18 shows, for 6-storey frame a very small reduction of seismic force occurred compared to a fixed-base response, while for the 3-storey frame lateral force increased by about 15%. Even though the base shears resemble to those of the fixed-base structure, the general responses quite different when SSI is included in the analysis. For example, the brace demands distributions and the lateral inter-story drift profile differ for two frame heights differ significantly as the rocking movement occurs when SSI is accounted for and play a significant role in overall behaviour.

### *Soil bearing demands*

A better response in the soil bearing demands was observed when foundations increased in size. A ratio of the maximum spring force and the ultimate bearing capacity of the soil ( $q_f/q_{ult}$ ) of 0.87 and 0.77 were recorded for 3- and 6-storeys frames respectively, indicating that the factored capacity of the soil was exceeded. Even if these values do not seem problematic, important inelastic displacements are expected according with the QzSimple2 backbone curve shown in Figure 4.18 and Figure 4.19. After reached a normalized load of 0.36 (value in which linear displacement is no longer reflected), significant inelastic displacements are triggered leading to relevant settlements.

### *Settlements, uplift and footing rotation*

Due to the inelastic behavior of the springs, the vertical movements of the footing are accentuated compared to the response on stiffer site class. Rotation of foundation was triggered when the overturning moment reached that calculated for  $R_d R_o$  equal to 3. Despite of important inelastic activity of the springs, 3-storeys scenario present acceptable values for settlements and rotations. A permanent net settlement smaller than 25 mm and a maximum differentials settlement smaller than 1/150. Note that the later value does not comprise the vertical movement due to tributary vertical load. Rotation obtained from the analysis were smaller than the design estimated by a rate of 0.40. For 6-storey frame, the settlements above 100 mm were still recorded. The value is seems not adequate for good performance of the building during an earthquake because it can lead to important structural and non-structural damage.

Table 5.15 Footing dimensions for scenarios SC32 and SC33.

| Scenario                   | Length | Width | Thickness |
|----------------------------|--------|-------|-----------|
| (--)                       | m      | m     | m         |
| SC32-6S-NBC20-VCR-E-CP-A23 | 22     | 12    | 2         |
| SC33-3S-NBC20-VCR-E-CP-A23 | 18     | 7     | 1.5       |

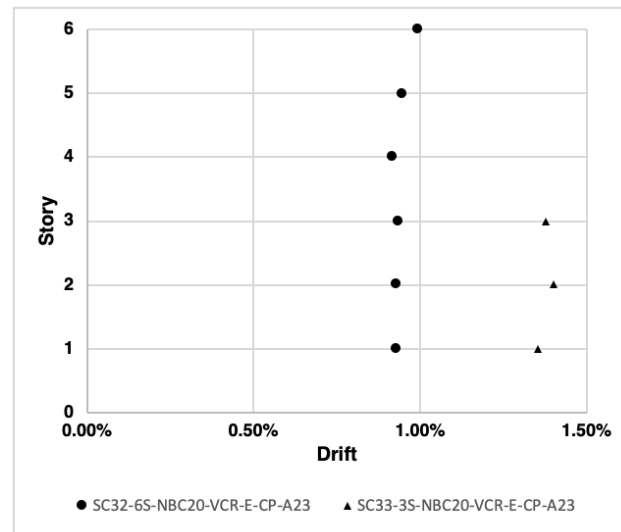


Figure 5.19 Inter-storey drift for scenarios SC32 and SC33.

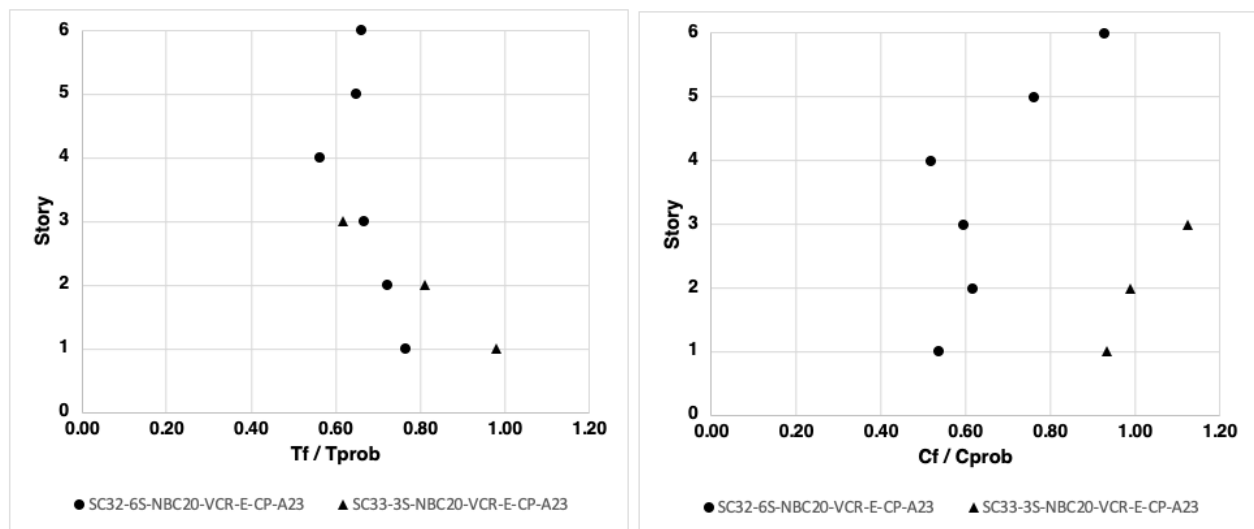


Figure 5.20 Diagonals demands for scenarios SC32 and SC33.

Table 5.16 Normalized developed overturning moment for scenarios SC32 and SC33.

| Scenario                   | Normalized moment to $M_{pr}$ |                |                |                |                    |
|----------------------------|-------------------------------|----------------|----------------|----------------|--------------------|
|                            | $M_{pr} / M_{pr,desig}$       | $M_f / M_{pr}$ | $M_d / M_{pr}$ | $M_n / M_{pr}$ | $M_{rot} / M_{pr}$ |
| SC32-6S-NBC20-VCR-E-CP-A23 | 1.17                          | 0.76           | 1.09           | 0.62           | 0.48               |
| SC33-3S-NBC20-VCR-E-CP-A23 | 1.14                          | 1.05           | 1.04           | 0.65           | 0.49               |

Table 5.17 Equivalent  $R_d R_o$  for overturning moment for scenarios SC32 and SC33.

| Scenario                   | Equivalent $R_d R_o$ |       |       |       |           |
|----------------------------|----------------------|-------|-------|-------|-----------|
|                            | $M_{pr}$             | $M_f$ | $M_d$ | $M_n$ | $M_{rot}$ |
| SC32-6S-NBC20-VCR-E-CP-A23 | 1.45                 | 1.91  | 1.33  | 2.33  | 3.00      |
| SC33-3S-NBC20-VCR-E-CP-A23 | 1.57                 | 1.40  | 1.41  | 2.24  | 3.00      |

Table 5.18 Base shear results for scenarios SC32 and SC33.

| Scenario                   | $V_f$<br>(kN) | Seismic<br>input<br>reduction |
|----------------------------|---------------|-------------------------------|
| SC32-6S-NBC20-VCR-E-CP-A23 | 5199          | 0.96                          |
| SC33-3S-NBC20-VCR-E-CP-A23 | 3641          | 1.16                          |

Table 5.19 Footing settlements and uplift summary for scenarios SC32 and SC33.

| Scenario                   | Settlement <sub>min</sub><br>(mm) | Uplift <sub>max</sub><br>(mm) | Permanent<br>(mm) | Permanent net<br>(mm) |
|----------------------------|-----------------------------------|-------------------------------|-------------------|-----------------------|
| SC32-6S-NBC20-VCR-E-CP-A23 | -127                              | 27                            | -127              | -117                  |
| SC33-3S-NBC20-VCR-E-CP-A23 | -42                               | 8                             | -24               | -19                   |

Table 5.20 Footing estimated rotations by A23.3 provisions and by non-linear analyses with SSI implemented for scenarios SC 32 and SC33.

| Scenario                   | $R_{A23.3}$ | $R_{OPENSEES}$ | $R_{OPENSEES} / R_{A23.3}$ |
|----------------------------|-------------|----------------|----------------------------|
| SC32-6S-NBC20-VCR-E-CP-A23 | 0.0123      | 0.0161         | 1.31                       |
| SC33-3S-NBC20-VCR-E-CP-A23 | 0.0164      | 0.0066         | 0.40                       |

## 5.12 Convergence challenges for Class E site

As discussed before, the Beam on Non-Linear Winkler Foundations by using QzSimple as spring material is a convenient modelling technique to account for the SSI in the analysis. The behavior of the soil can be approximated as well as its influence on the superstructure. However, their implementation requires the construction and calibration of complex models for which achieving convergence can be challenged, particularly for very soft soil conditions for which substantial displacements due to settlements and uplift can be expected.

Because soil conditions and expected response for site E meet these characteristics, some convergence difficulties were observed during the analyses. Same analyses parameters such as increment type, algorithms, integrators and convergence tolerances values were used for Class C and E sites. However, with parameters judged adequate for Class C site, it was not always possible to achieve convergence for Class E site models. In addition to convergence difficulties, some outliers values for spring forces were observed for certain E scenarios. More detailed examination of results permitted to conclude that the issues of convergence and outlier were in all cases related to the spring response. By the end of the current study, Madani (2023) developed a methodology to approach beforementioned challenges. After applying the suggested procedures, it was confirmed that only spring force response could have been affected (i.e larger than realistic spring forces might have been recorded), while all other response parameters remained unchanged. Additional investigations should be done in the future to fully assess these issues.

## **CHAPTER 6      CONCLUSION AND RECOMMENDATIONS**

In this chapter conclusions of this study are presented, along with recommendations and suggestions for the future work.

### **6.1 Summary**

NBCC acknowledge that the rocking of the foundation can occurs during an earthquake and has and influence into the response of the superstructure. Because of this, the foundation movements must be estimated and contemplated in the design of the SFRS. The procedure described by the Canadian Standard A23.3 recognises this behavior and propose two alternatives for the estimation of footing movements. Capacity Protected foundations seek to limit the rocking for the foundation using a design moment equal of greater than the capacity overturning moment of the SFRS. This consideration leads to a large footing dimension that limits the rocking. On the other hand, Not-Capacity Protected foundation procedures recognize that if the foundation dimensions cannot sustain the complete overturning capacity of the SFRS, additional rotation of the foundation can occur. The available methods exposed by A23.3 to estimate this rotation movements, were investigated in the scope of this study by employed some methods of soil-structure interaction. The study focused on foundation free-to-rotate, i.e the cases in which such behaviour is not possible (i.e. anchored foundations, piles, several underground basements etc.) are out of the scope of this study. Several factors that influence the response of the system soil-foundation-superstructure were studied. The height of the building, the site Class, the regional seismic hazard and the type of foundation are among the most important studied factors. To achieve the presented results, a suite of structural analysis where performed. Prototype building that reflect a typical Canadian office building were designed with the latest edition of NBC 2020, CSA S16-19 and CSA A23.3-19. Moderately ductile concentrically braced frames with X-type configuration were considered as SFRS. Diagonals were desgined as HSS sections with grade defined by the Canadian Standad G40.20. Foundation movements-related provisions were ignored in these designs with the purpose of investigate the rocking phenomenon. Numerical models of these designs were constructed by following some verified structural analysis procedures. Special attention was taken along the modeling and analysis techniques to properly reflect the actual behavior of a real building. Inelastic

response of braces was taken into account, second-order effects were considered, dimensions of connections were explicitly introduced into the models, calibration of SSI were performed to reflect the rocking response of the foundations. Footing sliding, local buckling nor rupture of the diagonals were not considered in the modeling. Finally, time-history structural analyses using ground motions records that reflect the regional seismic hazard were executed. The results of these simulations were processed in such manner to get structural parameter to compare between scenarios and between code limitations and tolerances.

The final results show, in overall terms, an adequate seismic response of the structure when site Class C was considered. Lateral displacements meet the limits imposed by the code. Structural performance of individual elements of the superstructure comply with design objectives. Soil bearing demands are below of the factored capacity of the soil. Settlements of foundation settlements results were small and permanent settlements were almost inexistent because the high rigidity of the soil. On the other hand, uplift always occurs, however, rotations of the foundation associated with the uplift were far below to those estimated with A23.3 proposed methods. As a result, it is judged adequate to do not increase the lateral displacement of the superstructure due to the footing rotation by the amount calculated with current code-imposed methods when site Class C is considered. Because of this excellent performance, further analyses with smaller foundations that do not meet any overstrength nor capacity-design provisions were accounted. When these considerations were taken, performance of the 3-storey buildings were at the very limit of an adequate response. 6-storey buildings do not meet the lateral displacement limits. These last results suggest that a minimum amount of overstrength consideration when designing foundations are favourable to accomplish an appropriated design that meet the seismic design objectives required by the NBCC and there is no necessity to increase lateral displacements to the superstructure due to foundation rocking.

Site Class E was also investigated to evaluate the influence of the soil rigidity into the overall rocking behavior. Results shows a significant different response compared with results observed in Site Class C. Extremely large settlements were observed. In some cases, up to 300 mm. Settlements along all the foundation were observed, rather than a rocking behavior. Yielding of the soil occurs along the length of the foundation leading to permanent settlements. In some cases, these permanent settlements were not uniform along the foundation, resulting in a tilted structure after the earthquake. These results suggest that even if the foundation were designed to sustain the

expected demands, due to the low rigidity of the soil, a larger foundation may be required. Additional simulations fully complying with code foundation-movement related provisions were performed. A better response with larger foundations were observed. Even though, if yielding still occurs, all other parameters that characterize the structure response show an acceptable response. In additions, convergence challenges were observed for buildings located in site E. Difficulties in achieve convergence and the occurrence of outliers were presented. A sensitivity study was referred to address these issues and it was found that different algorithms, integrators and converge tolerances can be employed to achieve converged. Similarly, a refined mesh for the implementation of SSI is advised when soft soil is considered.

## 6.2 Conclusions

1. Increase in seismic design loads were identified in the 2020 edition of NBC. However, the weight of the SFRS is not increased in the same rate.
2. The SSI can have a significant influence on the response of the superstructure. Indeed, a reduction in the frame demands has been observed as well as modifications in modal period, lateral displacements and soil demands. For stiffer soil, with increase of foundation dimensions the response of the system approached that of the system with a fixed base. When dimensions decrease, the rocking behaviour takes place which limit the seismic input into the superstructure and provide protection. The influence of SSI on sites E is also significant, however for the softer soil high forces in soil springs and an substantial settlements occurred rather than a rocking behaviour.
3. Even when the dimensions of the foundations are not increased to meet inter-storey drift requirements increased by footing rotation, for class C site, both frames exhibited a satisfactory response when SSI was included in the analysis. Total drifts did not exceed the limits imposed by the NBC. Brace tensile and compressive force demands were reduced and all performance parameters of the substructure remain with acceptable range.
4. For Class C site it was possible to achieve a satisfactory structural response when foundations were designed without any capacity considerations ( $RdRo = 3.9$ ). Rocking behaviour dominated the response. Total drift remained well under NBC limits for three-storey frame, but was very close to the limit for six-storey frame. The later implies a potential for non-structural and structural damage in the building. Brace force demands, both in compression and in tension were reduced and the all performance indicators of the

superstructure remained with acceptable range. Soil yielding occurred underneath the foundation edges, however, the overall response is judged acceptable for a seismic performance at collapse prevention level.

5. Foundations of CBFs designed by the current Canadian provisions for earthquake resistant applications on sites C could be considered as oversized based in the minimum objectives of earthquake resistant design.
6. Soil underneath the foundations designed without respecting the criterion of increased drift due to foundation rotations on Class E site did not show adequate response when SSI was accounted for in the analysis. When design is compliant with all design provisions, the response of the system soil-foundation frame exhibits an acceptable performance, particularly for the lower frames. However, caution is advised since some convergence issues were found during the non-linear analyses.

This study was done for two CBF frame heights in building with similar layouts and two site conditions. Before general conclusions can be drawn, other structural configurations with possibly different dynamic characteristics should be examined.

### **6.3 Recommendations**

Because of the sensitivity in the response when very soft site is considered, (i.e. site Class E) the examination with a wider range of geotechnical properties are advised.

Other approaches employed to set an acceptance criterion based in various levels of performance are suggested in order to relate the SSI and the structural damage due to inelastic rotation of foundations. This can be helpful to establish different levels of foundation design moments based in a desired target performance.

A limited number of cases and building configurations were investigated through this study. Further examination for different building configurations and geomechanics properties is advised before conclusions could be generalised.

### **6.4 Future work**

In order to complement the present study and reach more general conclusions, the following work is suggested in further:

Build a suitable Ground Motion Database for the East Canada and complete the analyses for Montreal scenarios.

Perform a set of pushover analyses in order to understand yielding sequence of the diagonals, triggers of foundation rotations, and identify foundation and soil response in more controlled analytical environment.

A comprehensive study of the reliability of the modeling and analysis procedures is suggested for scenarios on Class E site due observed convergence issues and possible outliers in the results.

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- +structures&spage=&date=2008-01-01&atitle=Nonlinear+winkler-based+shallow+foundation+model+for+performance+assessment+of+seismically+loaded+structures&au=Raychowdhury%2C+Prishati&id=doi: ProQuest Dissertations & Theses Global database. (3307689)
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## APPENDIX A GUSSET PLATE DIMENTIONS

In Table A.1 and Table A.2 show the final dimensions of the gusset plates are presented. In these tables t, Y and X values represent the thickness, the vertical dimension of the plate in contact with the column and the horizontal dimension of the plate in contact with the beam respectively. Lower location denotes the connection at level of the floor for a given storey while Upper location denotes the connection at the ceiling location of the same given storey.

Table A.1 Gusset plate final dimensions for 6-storey building scenarios.

| No. | Scenario       | Storey | Location | Brace         | t<br>(mm) | L <sub>w</sub><br>(mm) | Y<br>(mm) | X<br>(mm) |
|-----|----------------|--------|----------|---------------|-----------|------------------------|-----------|-----------|
| 1   | 6S-NBC20-MTL-C | 6      | Lower    | HS102X102X6.4 | 19        | 250                    | 72        | 742       |
| 2   | 6S-NBC20-MTL-C | 5      | Lower    | HS127X127X6.4 | 22        | 300                    | 123       | 843       |
| 3   | 6S-NBC20-MTL-C | 4      | Lower    | HS127X127X6.4 | 22        | 300                    | 129       | 937       |
| 4   | 6S-NBC20-MTL-C | 3      | Lower    | HS127X127X7.9 | 25        | 300                    | 134       | 945       |
| 5   | 6S-NBC20-MTL-C | 2      | Lower    | HS127X127X9.5 | 25        | 300                    | 137       | 941       |
| 6   | 6S-NBC20-MTL-C | 1      | Lower    | HS152X152X7.9 | 25        | 360                    | 367       | 694       |
| 7   | 6S-NBC20-MTL-C | 6      | Upper    | HS102X102X6.4 | 19        | 250                    | 71        | 745       |
| 8   | 6S-NBC20-MTL-C | 5      | Upper    | HS127X127X6.4 | 22        | 300                    | 123       | 843       |
| 9   | 6S-NBC20-MTL-C | 4      | Upper    | HS127X127X6.4 | 22        | 300                    | 151       | 893       |
| 10  | 6S-NBC20-MTL-C | 3      | Upper    | HS127X127X7.9 | 25        | 300                    | 134       | 945       |
| 11  | 6S-NBC20-MTL-C | 2      | Upper    | HS127X127X9.5 | 25        | 300                    | 138       | 939       |
| 12  | 6S-NBC20-MTL-C | 1      | Upper    | HS152X152X7.9 | 32        | 360                    | 199       | 1064      |
| 13  | 6S-NBC20-MTL-E | 6      | Lower    | HS102X102X7.9 | 19        | 250                    | 50        | 784       |
| 14  | 6S-NBC20-MTL-E | 5      | Lower    | HS127X127X9.5 | 32        | 300                    | 113       | 914       |
| 15  | 6S-NBC20-MTL-E | 4      | Lower    | HS152X152X9.5 | 32        | 355                    | 196       | 1070      |
| 16  | 6S-NBC20-MTL-E | 3      | Lower    | HS152X152X9.5 | 32        | 355                    | 196       | 1070      |
| 17  | 6S-NBC20-MTL-E | 2      | Lower    | HS152X152X13  | 35        | 400                    | 226       | 1120      |
| 18  | 6S-NBC20-MTL-E | 1      | Lower    | HS203X152X13  | 32        | 460                    | 469       | 885       |
| 19  | 6S-NBC20-MTL-E | 6      | Upper    | HS102X102X7.9 | 19        | 250                    | 70        | 745       |
| 20  | 6S-NBC20-MTL-E | 5      | Upper    | HS127X127X9.5 | 32        | 300                    | 114       | 912       |

Table A.1 Gusset plate final dimensions for 6-storey building scenarios (continue).

|    |                |   |       |               |    |     |     |      |
|----|----------------|---|-------|---------------|----|-----|-----|------|
| 21 | 6S-NBC20-MTL-E | 4 | Upper | HS152X152X9.5 | 32 | 355 | 196 | 1070 |
| 22 | 6S-NBC20-MTL-E | 3 | Upper | HS152X152X9.5 | 32 | 355 | 196 | 1070 |
| 23 | 6S-NBC20-MTL-E | 2 | Upper | HS152X152X13  | 35 | 355 | 200 | 1054 |
| 24 | 6S-NBC20-MTL-E | 1 | Upper | HS203X152X13  | 38 | 410 | 265 | 1199 |
| 25 | 6S-NBC20-VCR-C | 6 | Lower | HS102X102X6.4 | 19 | 250 | 72  | 742  |
| 26 | 6S-NBC20-VCR-C | 5 | Lower | HS114X114X7.9 | 22 | 300 | 94  | 873  |
| 27 | 6S-NBC20-VCR-C | 4 | Lower | HS127X127X9.5 | 25 | 300 | 132 | 944  |
| 28 | 6S-NBC20-VCR-C | 3 | Lower | HS127X127X13  | 32 | 300 | 141 | 962  |
| 29 | 6S-NBC20-VCR-C | 2 | Lower | HS178X127X9.5 | 32 | 355 | 210 | 1077 |
| 30 | 6S-NBC20-VCR-C | 1 | Lower | HS178X127X13  | 32 | 350 | 383 | 718  |
| 31 | 6S-NBC20-VCR-C | 6 | Upper | HS102X102X6.4 | 19 | 250 | 71  | 745  |
| 32 | 6S-NBC20-VCR-C | 5 | Upper | HS114X114X7.9 | 25 | 300 | 120 | 838  |
| 33 | 6S-NBC20-VCR-C | 4 | Upper | HS127X127X9.5 | 25 | 300 | 133 | 942  |
| 34 | 6S-NBC20-VCR-C | 3 | Upper | HS127X127X13  | 32 | 300 | 141 | 962  |
| 35 | 6S-NBC20-VCR-C | 2 | Upper | HS178X127X9.5 | 32 | 355 | 210 | 1077 |
| 36 | 6S-NBC20-VCR-C | 1 | Upper | HS178X127X13  | 32 | 350 | 206 | 1071 |
| 37 | 6S-NBC20-VCR-E | 6 | Lower | HS102X102X7.9 | 19 | 250 | 49  | 782  |
| 38 | 6S-NBC20-VCR-E | 5 | Lower | HS127X127X13  | 32 | 300 | 109 | 919  |
| 39 | 6S-NBC20-VCR-E | 4 | Lower | HS254X127X13  | 38 | 450 | 290 | 1380 |
| 40 | 6S-NBC20-VCR-E | 3 | Lower | HS254X127X13  | 38 | 450 | 286 | 1388 |
| 41 | 6S-NBC20-VCR-E | 2 | Lower | HS254X127X16  | 44 | 500 | 327 | 1440 |
| 42 | 6S-NBC20-VCR-E | 1 | Lower | HS254X152X16  | 38 | 500 | 532 | 996  |
| 43 | 6S-NBC20-VCR-E | 6 | Upper | HS102X102X7.9 | 19 | 250 | 70  | 741  |
| 44 | 6S-NBC20-VCR-E | 5 | Upper | HS127X127X13  | 32 | 300 | 113 | 911  |
| 45 | 6S-NBC20-VCR-E | 4 | Upper | HS254X127X13  | 38 | 450 | 319 | 1322 |
| 46 | 6S-NBC20-VCR-E | 3 | Upper | HS254X127X13  | 38 | 450 | 290 | 1380 |
| 47 | 6S-NBC20-VCR-E | 2 | Upper | HS254X127X16  | 44 | 500 | 327 | 1440 |
| 48 | 6S-NBC20-VCR-E | 1 | Upper | HS254X152X16  | 51 | 500 | 336 | 1458 |

Table A.2 Gusset plate final dimensions for 3-storey building scenarios.

| No. | Scenario       | Storey | Location | Brace         | t<br>(mm) | L <sub>w</sub><br>(mm) | Y<br>(mm) | X<br>(mm) |
|-----|----------------|--------|----------|---------------|-----------|------------------------|-----------|-----------|
| 1   | 3S-NBC20-MTL-C | 3      | Lower    | HS102X102X6.4 | 19        | 250                    | 119       | 728       |
| 2   | 3S-NBC20-MTL-C | 2      | Lower    | HS114X114X9.5 | 25        | 300                    | 142       | 876       |
| 3   | 3S-NBC20-MTL-C | 1      | Lower    | HS127X127X13  | 32        | 350                    | 352       | 598       |
| 4   | 3S-NBC20-MTL-C | 3      | Upper    | HS102X102X6.4 | 19        | 250                    | 119       | 728       |
| 5   | 3S-NBC20-MTL-C | 2      | Upper    | HS114X114X9.5 | 25        | 300                    | 142       | 876       |
| 6   | 3S-NBC20-MTL-C | 1      | Upper    | HS127X127X13  | 32        | 350                    | 176       | 912       |
| 7   | 3S-NBC20-MTL-E | 3      | Lower    | HS102X102X9.5 | 22        | 250                    | 124       | 731       |
| 8   | 3S-NBC20-MTL-E | 2      | Lower    | HS127X127X13  | 32        | 300                    | 137       | 944       |
| 9   | 3S-NBC20-MTL-E | 1      | Lower    | HS152X152X13  | 35        | 350                    | 372       | 625       |
| 10  | 3S-NBC20-MTL-E | 3      | Upper    | HS102X102X9.5 | 22        | 250                    | 124       | 731       |
| 11  | 3S-NBC20-MTL-E | 2      | Upper    | HS127X127X13  | 32        | 300                    | 137       | 944       |
| 12  | 3S-NBC20-MTL-E | 1      | Upper    | HS152X152X13  | 35        | 350                    | 195       | 939       |
| 13  | 3S-NBC20-VCR-C | 3      | Lower    | HS102x102x6.4 | 19        | 250                    | 119       | 731       |
| 14  | 3S-NBC20-VCR-C | 2      | Lower    | HS127x127x7.9 | 32        | 300                    | 159       | 907       |
| 15  | 3S-NBC20-VCR-C | 1      | Lower    | HS152x152x7.9 | 35        | 375                    | 387       | 664       |
| 16  | 3S-NBC20-VCR-C | 3      | Upper    | HS102x102x6.4 | 19        | 250                    | 119       | 731       |
| 17  | 3S-NBC20-VCR-C | 2      | Upper    | HS127x127x7.9 | 32        | 300                    | 159       | 907       |
| 18  | 3S-NBC20-VCR-C | 1      | Upper    | HS152x152x7.9 | 35        | 375                    | 234       | 937       |
| 19  | 3S-NBC20-VCR-E | 3      | Lower    | HS102x102x7.9 | 19        | 250                    | 124       | 740       |
| 20  | 3S-NBC20-VCR-E | 2      | Lower    | HS127x127x13  | 32        | 300                    | 164       | 916       |
| 21  | 3S-NBC20-VCR-E | 1      | Lower    | HS152x152x13  | 35        | 350                    | 376       | 643       |
| 22  | 3S-NBC20-VCR-E | 3      | Upper    | HS102X102X7.9 | 19        | 250                    | 124       | 740       |
| 23  | 3S-NBC20-VCR-E | 2      | Upper    | HS127x127x13  | 32        | 300                    | 164       | 916       |
| 24  | 3S-NBC20-VCR-E | 1      | Upper    | HS152x152x13  | 35        | 350                    | 200       | 957       |

## APPENDIX B STKO MODELS

This appendix shows figures of the STKO models of the CBFs and some of images of the post-processing visualization.

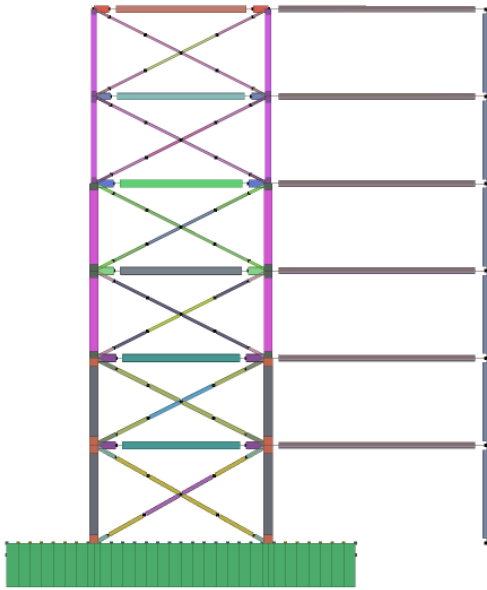


Figure 6.2 SC09-6S-NBC20-VCR-C-NCP

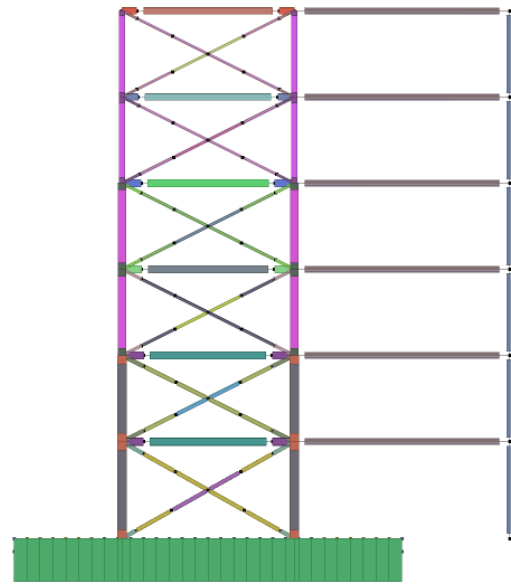


Figure 6.1 SC08-6S-NBC20-VCR-C-CP

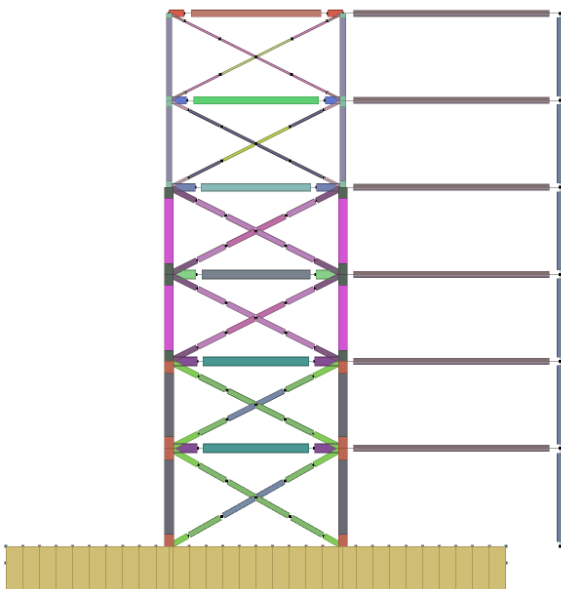


Figure 6.3 SC11-6S-NBC20-VCR-E-CP

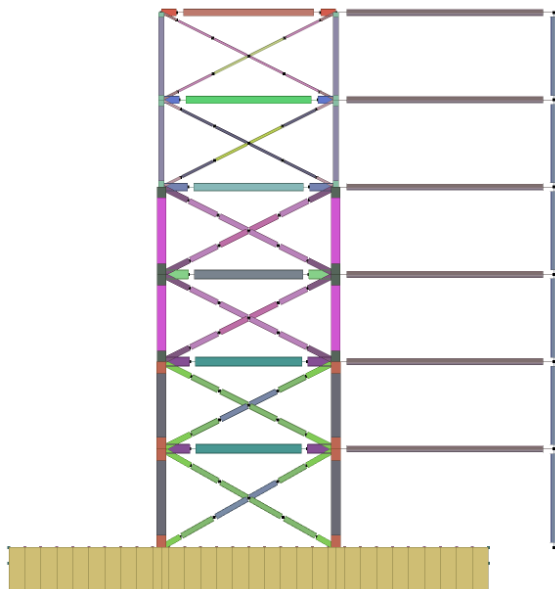


Figure 6.4 SC12-6S-NBC20-VCR-E-NCP

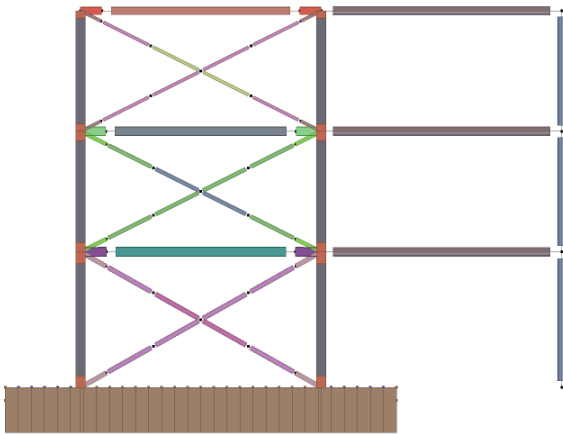


Figure 6.8 SC21-3S-NBC20-VCR-C-NCP

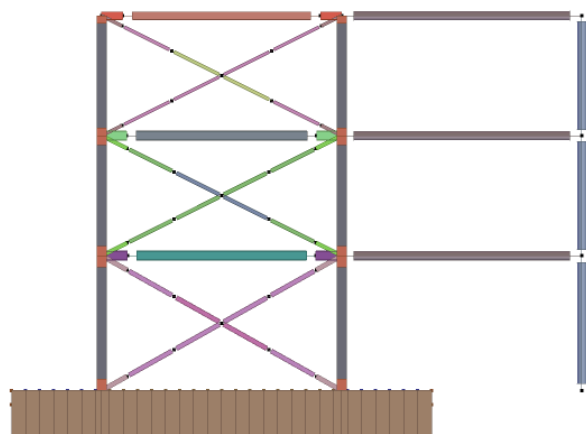


Figure 6.5 SC20-3S-NBC20-VCR-C-CP

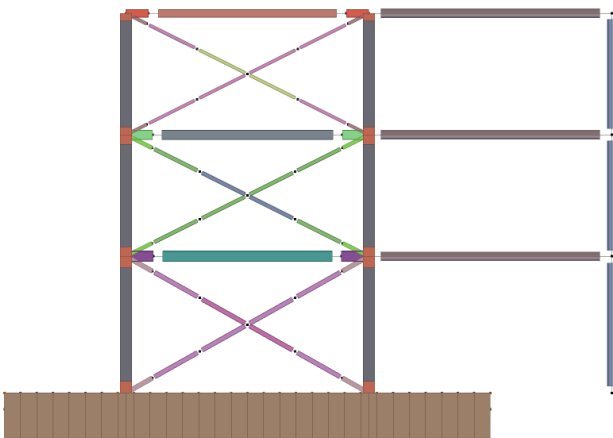


Figure 6.7 SC24-3S-NBC20-VCR-E-NCP

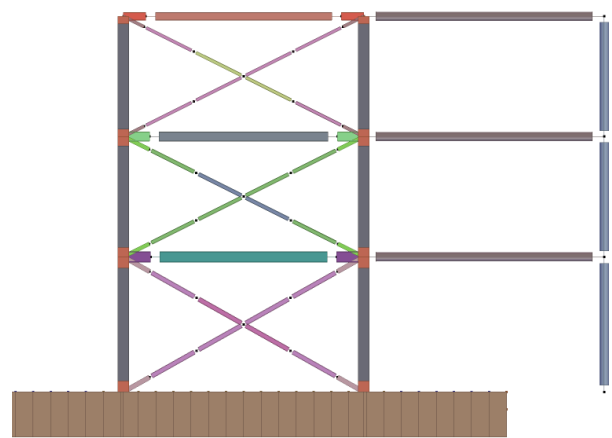


Figure 6.6 SC23-3S-NBC20-VCR-E-CP

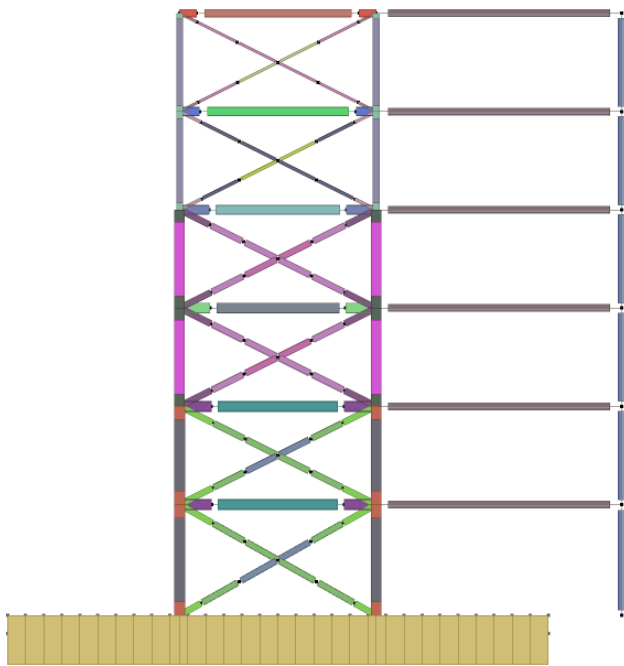


Figure 6.11 SC31-3S-NBC20-VCR-C-RdRo=3.9

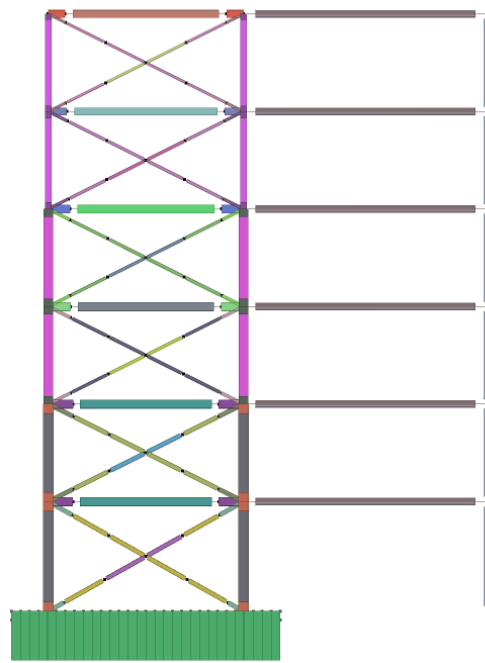


Figure 6.10 SC30-6S-NBC20-VCR-C-RdRo=3.9

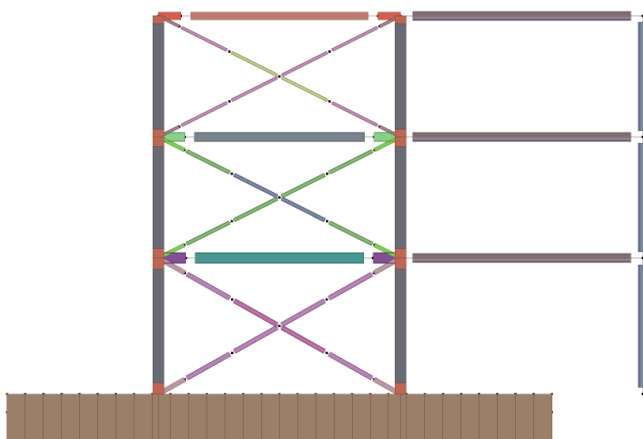


Figure 6.12 SC33-3S-NBC20-VCR-E-CP-A23

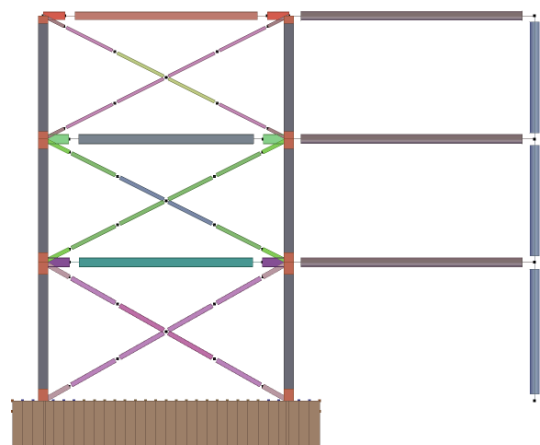


Figure 6.9 SC32-6S-NBC20-VCR-E-CP-A23

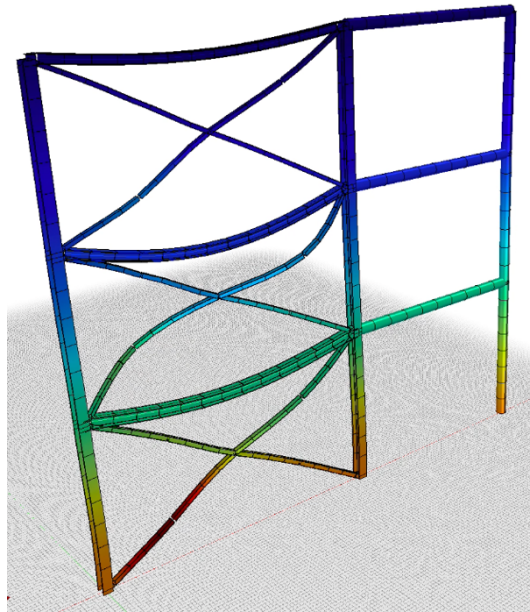


Figure 6.13 Out-of-plane flexural buckling of the braces captured during the analysis. Note that visualization is scale up for illustration purposes.

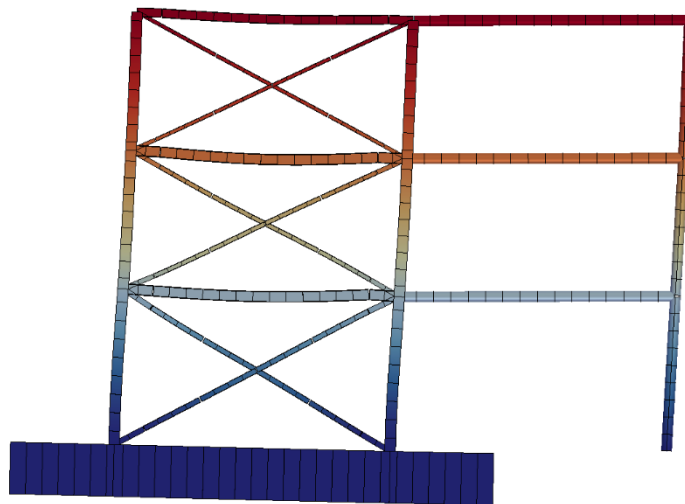


Figure 6.14 Uplift of the foundation captured during the analysis. Note that visualization is scale up for illustration purposes.