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affiliée à l'Université de Montréal

**The effect of soil-structure interaction on seismic design of eccentrically
braced frames foundations**

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

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présenté par **Bardia RAMIN AFSHAR**

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DEDICATION

I dedicate this dissertation to all the beloved individuals who have surrounded and supported me throughout my academic journey. Your unwavering encouragement, guidance, and belief in me have been the driving force behind my accomplishments. This work serves as a tribute to the collective love, strength, and inspiration you have graciously given me.

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RÉSUMÉ

La relation entre le basculement des fondations et la conception de la capacité est essentielle dans la conception des fondations d'une structure en acier, particulièrement dans les zones sujettes à l'activité sismique. Ces concepts jouent un rôle déterminant pour améliorer les performances sismiques des bâtiments, assurant leur résilience et leur sécurité en cas de tremblement de terre. La fondation oscillante implique une conception délibérée de la base du bâtiment pour permettre un mouvement contrôlé lors des événements sismiques. Contrairement aux fondations fixes traditionnelles, qui transmettent directement les forces sismiques à la structure, entraînant potentiellement des dommages importants, une fondation oscillante permet au bâtiment de se déplacer d'avant en arrière sur sa base, réduisant ainsi les forces latérales transmises à la superstructure.

En réalité, une partie de l'énergie sismique est dissipée par l'oscillation. Cependant, la norme CSA A23.3-19 se concentre sur la création de fondations favorisant ce comportement oscillant, sans donner de détails précis sur les charges sismiques requises pour concevoir les fondations des ossatures en acier contreventées. Cette procédure de conception de fondation est spécifiquement adaptée aux systèmes de résistance sismique en béton. Les demandes et les facteurs de sécurité mentionnés dans la norme canadienne de conception de l'acier reposent sur des études portant sur les murs de refends en béton.

La procédure de conception de fondation pour les systèmes de murs de cisaillement en béton est basée sur les charges issues des exigences de ductilité à l'intersection entre la fondation et les murs de cisaillement. Cependant, dans les ossatures en acier contreventées, les demandes de ductilité proviennent du mécanisme de plasticité présent dans tous les éléments de la structure, engendrant ainsi une charge cumulative sur la fondation. Ceci peut conduire à des estimations de conception excessivement prudentes pour les besoins de la fondation et les déplacements totaux du bâtiment. Cette approche affecte négativement non seulement les fondations elles-mêmes, en augmentant leur taille et leur coût, mais également le choix de l'acier comme matériau pour le système de résistance sismique. Cette recherche examinera cette problématique en utilisant une méthode d'analyse non linéaire.

Pour valider la norme CSA A23.3-19, deux immeubles de bureaux, un de 3 étages et l'autre de 8 étages, ont été pris en compte dans deux zones sismiques majeures, à Montréal et à Vancouver. En

outre, pour étendre les résultats en raison de la diversité des sols, des sites de sol de classe C et E ont été sélectionnés pour représenter respectivement des bases rigides et flexibles. Des cadres en acier avec contrefiche excentrique, présentant un $R_d R_o$ de 6, ont été conçus selon le principe de conception de capacité de la norme CSA S16-19 pour chaque emplacement et classe de sol.

Trois types distincts de bases sont pris en considération : base fixe, base protégée par capacité (CP) et sans facteur de capacité (NCF). Le scénario de base fixe se réfère à des conditions où l'interaction sol-structure n'a aucune influence sur le moment de renversement. En général, l'interaction sol-structure a tendance à réduire la demande sur la fondation. Ainsi, l'utilisation du scénario de base fixe fournit une estimation conservatrice, représentant une limite supérieure pour le moment de renversement sur la fondation. Ensuite, la fondation CP est spécifiquement conçue pour correspondre aux moments qui sont transmis via le V (probable) des liens de cisaillement. Par la suite, la fondation NF doit être alignée sur le moment qui découle de la charge sismique, où $V = \frac{S(T_d)M_v I_E W}{(R_d R_o = 6)}$.

En incorporant l'interaction sol-structure, un modèle numérique 2D a été créé à l'aide du logiciel STKO, connu sous le nom de "The Scientific ToolKit for OpenSees", pour l'analyse des cadres en acier avec contrefiche excentrique. Les liens ductiles, les poutres et les contrefiches ont été modélisés à l'aide d'éléments non linéaires avec des sections fibreuses capables de subir une plasticité. Pour modéliser le comportement en cisaillement des liens ductiles, "section aggregator" a été utilisé, permettant le développement de la plasticité de cisaillement dans la section. Les colonnes ont été modélisées à l'aide d'éléments élastiques qui maintiennent leur élasticité pendant les cycles de chargement et de déchargement. Les plaques de renfort ont été modélisées à l'aide d'éléments de ressort de longueur nulle. L'interaction sol-structure a été simulée à l'aide de la méthode de « Beam On Non-linear Winkler Foundation (BNWF) ». La fondation a été représentée comme une poutre élastique, et les caractéristiques du sol ont été représentées à l'aide d'éléments de ressort de longueur nulle, qui peuvent capturer le statut oscillant ainsi que le soulèvement ou le tassement du sol.

Pour l'ouest du Canada, 15 mouvements du sol ont été sélectionnés et calibrés en fonction des décompositions du spectre cible pour Vancouver et de la plage de périodes des cadres en acier avec contrefiche excentrique. Les enregistrements sismiques pour différents types de séisme (Crustal, Inslab, Interface) ont été extraits d'une gamme de bases de données. Pour l'est du Canada, 11

accélérogrammes simulés calibrés provenant de la base de données d'Atkinson ont été sélectionnés. Une analyse chronologique non linéaire a été réalisée sur les modèles représentant les systèmes comprenant la superstructure, la fondation et le sol, ce qui a permis d'évaluer les comportements des systèmes.

Les résultats montrent que, pour les fondations sur un site de classe C, même sans prise en compte des principes de conception par capacité, les limites de déplacement inter-étages et de capacité du sol ont été respectées. Cette méthode de conception a permis de réduire la taille des fondations ainsi que la demande en ductilité sur la superstructure. Comme prévu, la fondation oscillante a dissipé l'énergie sismique, entraînant des rotations inélastiques plus faibles des liens ductiles, mais elle a augmenté le déplacement inter-étages et les contraintes dans le sol.

ABSTRACT

The relationship between foundation rocking and capacity design is essential in the design of a steel frame structure's foundation, particularly in areas prone to seismic activity. These concepts play a crucial role in improving the seismic performance of buildings, ensuring their resilience and safety during earthquakes. Foundation rocking involves intentionally designing the building's foundation to permit controlled movement during seismic events. Traditional fixed foundations tend to transmit seismic forces directly to the structure, potentially leading to severe damage. However, with a rocking foundation, the building can sway back and forth on its base, reducing the lateral forces transmitted to the superstructure.

A portion of seismic energy is dissipated through the rocking motion. Consequently, the CSA A23.3-19 standard focuses on establishing foundations conducive to rocking behaviour, yet it lacks precise specifications regarding seismic load criteria for designing foundations specifically tailored to steel braced frames. This foundation design procedure is primarily intended for concrete seismic force-resisting systems. The requirements and safety factors outlined in the Canadian steel design standard are derived from research conducted on concrete shear walls.

The foundation design procedure for concrete shear wall systems is based on loads stemming from ductility demands at the intersection of the foundation and shear walls. Conversely, in braced steel frames, ductility demands originate from plasticity mechanisms present in all frame elements, resulting in cumulative loads on the foundation. This might lead to overly conservative estimations for foundation requirements and overall building drifts, negatively impacting not only the foundations themselves by necessitating increased size and cost but also influencing the selection of steel as the material for the force-resisting system. This research will address this issue through a nonlinear analysis method.

As to validate the CSA A23.3-19 standard two 3-and 8-storey office building are considered for two important seismic zone Montreal and Vancouver. Moreover, to broad the results due to the soil quality, site class C and E were selected to act as rigid and flexible bases. Steel eccentrically braced frame with $R_d R_o$ of 6 were designed according to the capacity design principle of CSA S16-19 standard for each of the locations and soil classes.

Four distinct types of bases are being considered: fixed base, Capacity-protected (CP), Not-capacity protected (NCP) and Non-capacity factored $R_d R_o=6$ (NCF). The first results on foundation

overturning moments confirmed that the not-capacity-protected foundation is not applicable in this research. The fixed base scenario refers to conditions that soil-structure interaction has no influence on the overturning moment. In general, soil-structure interaction tends to decrease the demand on the foundation. Thus, using the fixed base scenario provides a conservative estimate, representing an upper limit for the overturning moment on the foundation. Next, the CP foundation is specifically designed to correspond with the moments that are transmitted via the shear capacity of the ductile links. Subsequently, the foundation without any capacity design requirements should be aligned with the moment that arises from the earthquake load, where $V = \frac{S(T_d)M_v I_E W}{(R_d R_0 = 6)}$.

Incorporating soil-structure interaction, a 2D numerical model was created using the STKO software, known as The Scientific Toolkit for OpenSees, for the analysis of steel eccentrically braced frames. The ductile links, beams, and braces were modeled using nonlinear elements with fiber sections capable of undergoing plasticity. To model the shear behaviour of ductile links, the section aggregator was employed, allowing the development of shear plasticity in the section. The columns were modeled using elastic elements that maintain their elasticity during loading and unloading cycles. Gusset plates were modeled using zero-length spring elements. Soil-structure interaction was simulated using the beam on Nonlinear Winkler Foundation (BNWF) substructure method. The foundation was represented as an elastic beam, and the soil's characteristics were depicted using zero-length spring elements, which can capture the rocking status as well as the uplift or settlement of the soil.

For Western Canada, 15 ground motions were selected and calibrated based on the deaggregations of the design spectrum for Vancouver and the period range of steel eccentrically braced frames. The earthquake records for various types of seism (crustal, in-slab, Interface) were extracted from a range of databases. For Eastern Canada 11 calibrated simulated accelerograms from Atkinson's database were selected. Nonlinear time history analysis was conducted on the models that portray the systems comprising superstructure, foundation, and soil, resulting in an evaluation of the systems' behaviours.

The results show that for the foundations on competent soil (site class C) even when no capacity principles were considered in design, the design limits for inter-storey drift and soil capacity were met. This approach to design led to smaller foundations, as well as to a reduction of ductility demand on the superstructure. As anticipated, the rocking foundation dissipated seismic energy

and resulted in smaller inelastic link rotations, but it increased the inter-storey drift and the soil stresses.

TABLE OF CONTENTS

DEDICATION	III
ACKNOWLEDGEMENTS	IV
RÉSUMÉ.....	V
ABSTRACT	VIII
TABLE OF CONTENTS	XI
LIST OF TABLES	XV
LIST OF FIGURES.....	XVIII
LISTE OF SYMBOLS AND ABBREVIATIONS	XXIV
LIST OF APPENDICES	XXXIII
CHAPTER 1 INTRODUCTION.....	1
1.1 Context	1
1.2 Objectives.....	4
1.3 Methodology	4
1.4 Organization of the dissertation	5
CHAPTER 2 LITERATURE REVIEW.....	6
2.1 Seismic design.....	6
2.2 Superstructure.....	7
2.2.1 Loads and analysis methods.....	7
2.2.2 Capacity design	15
2.2.3 Eccentrically braced frame.....	16
2.2.4 Link beam.....	16
2.3 Foundation.....	26
2.3.1 Foundation provisions according to NBCC 2020	26

2.3.2	Shallow foundation	30
2.3.3	Rocking foundation	31
2.4	Soil-structure interaction	34
2.4.1	Definition of soil-structure interaction	35
2.4.2	Interaction mechanisms	35
2.4.3	Beam On Non-linear Winkler Foundation (BNWF)	38
2.4.4	Kinematic interaction	39
2.4.5	Inertial interaction	39
2.4.6	The effect of soil-structure interaction	43
2.5	Numerical modeling	45
2.5.1	STKO	45
2.5.2	Superstructure modeling	45
CHAPTER 3	PROTOTYPE BUILDING DESIGN	49
3.1	Building description	49
3.2	Design assumptions	50
3.3	Seismic hazard values	52
3.4	Calculation of T_{emp} , $S(T_a)$, V_{ESF} , V_{DYN}	53
3.5	Frame design	55
3.5.1	Ductile link	55
3.5.2	Other frame members	56
3.6	Design summary	56
3.7	Foundation design	59
3.7.1	Soil properties	59
3.7.2	Foundation demand	60

3.7.3	Summary of footing size	61
CHAPTER 4 NUMERICAL MODELING		62
4.1	Modeling of the ductile link	63
4.2	Modeling of beams out of the links and braces	66
4.3	Soil-structure interaction modeling	70
4.3.1	Beam-on-Nonlinear-Winkler	72
4.3.2	The QzSimple1	73
4.4	Ground Motions	86
4.4.1	Selection and Scaling of Ground Motions	88
4.4.2	Deaggregation of mean hazard for Montreal and Vancouver	91
4.4.3	Selection of accelerogram for Vancouver site class E and C	94
4.4.4	Scaling of Ground Motion Records	96
CHAPTER 5 RESULTS AND DISCUSION		109
5.1	Results for Vancouver	109
5.1.1	General comments	109
5.1.2	Periods of vibration	109
5.1.3	Lateral displacement	111
5.1.4	Overturning moment	114
5.1.5	Link inelastic rotation	116
5.1.6	Shear yielding capacity of ductile links	119
5.1.7	Soil bearing demands	122
5.1.8	Settlements and uplift	124
5.1.9	Footing rotation	127
5.1.10	Evaluation of the response of outer beam segments, braces and columns	128

5.2	Results for Montréal.....	134
5.2.1	General comments.....	134
5.2.2	Period of vibration.....	134
5.2.3	Lateral displacement	135
5.2.4	Overtuning moment	137
5.2.5	Link inelastic rotation.....	139
5.2.6	Shear yielding capacity of ductile links	142
5.2.7	Settlements and uplift.....	144
CHAPTER 6	CONCLUSION AND RECOMMENDATIONS.....	147
6.1	Summary	147
6.2	Conclusions	149
6.3	Recommendations	150
REFERENCES.....		151
APPENDIX A	STKO MODELS AND DIAGRAMS	156
APPENDICES.....		156

LIST OF TABLES

Table 2-1 Equations for determining shallow foundation stiffness after Gazetas (1991a) as summarized in ATC-40 (1996).	44
Table 3-1 Dead and Live loads for Montreal and Vancouver.....	51
Table 3-2 The seismic hazard values NBCC 2020 for Vancouver and Montreal.....	52
Table 3-3 Comparing between the seismic hazard values of NBCC 2020 and NBCC 2015	52
Table 3-4 The ratio between seismic shear force and the total weight of the building.....	53
Table 3-5 Calibration factor	54
Table 3-6 Section design for Montreal.....	57
Table 3-7 Section design for Vancouver.....	58
Table 3-8 Comparison between M probable and M corresponding rocking limit ($R_d R_o = 2$)	59
Table 3-9 Design Soil properties.....	60
Table 3-10 Overturning moment demand	60
Table 3-11 Footing dimensions for Vancouver 8-and 3-storey site class C and E	61
Table 4-1 Mean quadratic error values ($\%P_y$) (Aguero et al. 2006)	68
Table 4-2 QzSimple1 and QzSimple2 parameters	77
Table 4-3 Foundation vertical and rotational stiffness for different scenario in Vancouver.	84
Table 4-4 Soil spring properties for different scenarios in Vancouver	84
Table 4-5 Foundation vertical and rotational stiffness for different scenario in Montreal.	85
Table 4-6 Soil spring properties for different scenarios in Montreal.	85
Table 4-7 The period range of interest for 3-and 8-storey structure in Montreal and Vancouver were computed.	90
Table 4-8 Deaggregation of mean hazard for Montreal and Vancouver.....	93
Table 4-9 The average magnitude-distance (M-R) value resulting from the period range for two scenarios.	93

Table 4-10 selected ground motions.	94
Table 4-11- Selected ground motion for Vancouver site class C.....	95
Table 4-12 Selected ground motion for Vancouver site class E.	96
Table 4-13 Scale factor for Vancouver site class C, 3 and 8 storey buildings.....	97
Table 4-14 Scale factor for Vancouver site class E, 3-and 8-storey buildings.	100
Table 4-15 magnitude-distance scenario for each event type.	103
Table 4-16 Dividing the period range according to earthquake type site class C.	103
Table 4-17 Second scale factor for Vancouver 8-storey Class C site.	105
Table 4-18 Dividing the period range according to earthquake for site class E.	106
Table 4-19 Second scale factor for Vancouver 8-storey site class E.	108
Table 5-1 Period change according to different site class and foundations.	110
Table 5-2 Inter-storey drift percentage for 3-storey structure Class C and E sites.	111
Table 5-3 Inter-storey drift percentage for 8-storey structure site class C and E.....	113
Table 5-4 Overturning moment for different type of foundations	114
Table 5-5 Inelastic rotations for 3-storey building in Vancouver on site class C and E.....	116
Table 5-6 Inelastic rotations for an 8-storey building in Vancouver on site class C and E	118
Table 5-7 The ratio of Ω for 3 storey structure site class C and E.....	119
Table 5-8 The ratio of Ω for 8 storey structure soil C and E.....	121
Table 5-9 Foundation rotation on soil class C and E	127
Table 5-10 The proportion of the moment transferred by the link to other sections (VC1)	128
Table 5-11 The proportion of the moment transferred by the link to other sections (VC20)	129
Table 5-12 Interaction of moment and axial force for beams outside of the link (VC1).....	130
Table 5-13 Interaction of moment and axial force for beams outside of the link (VC20).....	130
Table 5-14 Interaction of moment and axial force for braces (VC1).....	131

Table 5-15 Interaction of moment and axial force for braces (VC20).....	132
Table 5-16 Column's first-level moment and axial force in Vancouver's 8-storey building on site Class C.....	133
Table 5-17 Period change for different types of soil and foundations (Montreal).....	134
Table 5-18 Inter-storey drift index for 3-storey frame site Class C and E.....	135
Table 5-19 Inter-storey drift percentage for 8-storey structure site class C and E (Montreal). ..	136
Table 5-20 Overturning moment for different type of foundations (Montreal).	138
Table 5-21 Inelastic rotations for an 3-storey building in Montreal on site class C and E	140
Table 5-22 Inelastic rotations for 8-storey building in Montreal on site class C and E.....	141
Table 5-23 The ratio of Ω for 3 storey structure on site class C and E	142
Table 5-24 The ratio of Ω for 8 storey structure site class C and E	143

LIST OF FIGURES

Figure 2-1: Decline of lateral resistance of the building caused by P- Δ effects (Tremblay 2021).	8
Figure 2-2 : Graphic representation of R_d and R_o factors (Tremblay 2021).	10
Figure 2-3 : The comparison of design spectrum for Montreal site category C increased between NBCC 2020 and NBCC 2015.	11
Figure 2-4 : The comparison of design spectrum for Vancouver site category C increased between NBCC 2020 and NBCC 2015.	11
Figure 2-5 : Lateral static seismic forces distribution (Tremblay 2021).	12
Figure 2-6 : Eccentrically braced frame (Nagui William 2019)	16
Figure 2-7 : Failure mechanisms of (a) short, (b) intermediate, and (c) long links (T. Okazaki 2005).	17
Figure 2-8 : Ductile segment (Tremblay 2021).	18
Figure 2-9 :Inelastic link rotation capacity for the UTA 9 specimen (Okazaki, T. 2005)	20
Figure 2-10 : The plastic rotation in DS changes according to the length of the link. (Tremblay 2021).	20
Figure 2-11 : Link overstrength factors reported in different experimental studies. (Sina Kazemzadeh Azad, 2017).	22
Figure 2-12 : Equivalent overturning moment reaction	27
Figure 2-13 : Stress distribution under shallow foundation. (Moehle 2015)	29
Figure 2-14 : The depth of a shallow foundation is less than its width $D < B$.	31
Figure 2-15 Soil-structure interaction (Lemieux 2014)	34
Figure 2-16 Structure-to-soil stiffness ratio for studies scenarios.	36
Figure 2-17 The significant impact of soil flexibility on a coupled CBF can change the demand-to-capacity ration of structural elements. Adapted from FEMA (2019).	36
Figure 2-18 Schematic illustration of DAA using continuum modeling by finite elements.	37
Figure 2-19 Substructure approach using springs that characterize the soil properties. Adapted	38

Figure 2-20 Total interaction is given as the sum of kinematic interaction and inertial interaction.	40
Figure 2-21 : Schematic illustration of deflections caused by force applied to: (a) fixed-base structure; and (b) structure with vertical, horizontal, and rotational flexibility at its base. (NEHRP 2012)	41
Figure 2-22 Concentrated and distributed plasticity. (NEHRP 2010)	46
Figure 2-23 FBE solution compared with the exact solution. (Terzic 2011)	47
Figure 2-24 DBE solution compared with the exact solution. (Terzic 2011)	47
Figure 2-25 Modeling of a nonlinear element (Mazzoni et al. 2007)	48
Figure 3-1 Building lay-out (Tremblay 2017 and Murugananthan 2018)	50
Figure 3-2 : Location of EBFs for Montreal Scenarios (a) and for Vancouver scenarios (b).....	50
Figure 3-3 Distribution of forces between the bracings including the effect of accidental torsion. (Tremblay 2022).....	54
Figure 3-4 Shear force and moment in the ductile link (Tremblay 2022).....	55
Figure 4-1 Numerical model OpenSees	62
Figure 4-2 Ductile link proposed by Richards and Uang (2003)	64
Figure 4-3 Simulation of ductile segment behaviour. (Ramadan and Ghobarah, 1995).....	64
Figure 4-4 Calibration of the link against experimental data. (T.Y. Yang et al, 2020)	66
Figure 4-5 Modeling of nonlinear Beam Column element. (Rozon 2008)	67
Figure 4-6 Computational time vs number of element, n_e and the errors. (Agüero et al, 2006)...	68
Figure 4-7 Experimental and numerical results of parametric assessment. (Agüero et al, 2006).	69
Figure 4-8 The damping curve defined by STKO software.....	70
Figure 4-9 Different mode of foundation deformation. (Raychowdhury, 2008)	71
Figure 4-10 BNWF Mesh discretization with variable stiffness intensity (Harden et al., 2005)..	72

Figure 4-11 Winkler model. a) foundation geometry b) OpenSees modeling. (Raychowdhury 2008)	73
Figure 4-12 QzSimple1, PySimple1 and TzSimple1 material models used in conjunction with zero Length element (Raychowdhury 2008).....	74
Figure 4-13 QzSimple1 backbone curve: (a) backbone curves used for soil-pile modeling and (b) general backbone curve (Raychowdhury 2008).....	74
Figure 4-14 Cyclic response of QzSimple1 (Boulanger, 2000b)	75
Figure 4-15 Distributed springs with vertical stiffness. (ASCE41 2013)	77
Figure 4-16 Footing geometric parameters. (NEHRP 2012)	78
Figure 4-17 Normalized end length versus footing aspect ratio given by (Harden et al., 2005) along with ATC-40 (1996) recommended value. (Raychowdhury 2008)	79
Figure 4-18 Increased end stiffness (a) spring distribution and (b) stiffness intensity ratio versus footing aspect ratio (Gajan 2007).....	80
Figure 4-19 BNWF model with stiffer springs at the end regions. (NEHRP 2012)	80
Figure 4-20 Effect of spring spacing on overall footing response: (a)–(b) moment versus rotation and (c)–(d) settlement versus rotation. (Raychowdhury 2008).....	82
Figure 4-21 Effect of spring spacing on normalized settlement demand. (Raychowdhury 2008)	83
Figure 4-22 System Soil-Column-Foundation. OpenSees modeling of the system. (Raychowdhury and Hutchinson 2008)	83
Figure 4-23 Map of seismicity in Canada to 2010, delineating the eastern and western seismic regions and the low seismicity central continental region. (user's guide NBC 2015).....	87
Figure 4-24 period range T_R (user's guide NBC 2015).....	89
Figure 4-25 Deaggregation of Montreal PGA for a probability of 2%/50 years. Numerical values in the upper right box allow for an analytical examination (Halchuk et al., 2007).....	91
Figure 4-26 Deaggregation of Vancouver for a probability of 2%/50 years (NBCC2015) for $S_a(0.2)$. Contributions from various earthquake sources are indicated in the figure. (Stephen Halchuk et al. 2019)	92

Figure 4-27 Compare of Unscaled ground motions with Vancouver hazard spectrum Class C site	98
Figure 4-28 Scaled accelerograms for 3-storey building located in Vancouver Class C site	98
Figure 4-29 First Scaled accelerograms for 8-storey building located in Vancouver Class C site	99
Figure 4-30 Second calibration to reduce the deviation of mean spectrum less than 10% with target spectrum for Vancouver 8-storey Class C site	99
Figure 4-31 Compare of Unscaled ground motions with Vancouver hazard spectrum Class E site	101
Figure 4-32 Scaled accelerograms for 3-storey building located in Vancouver Class E site	101
Figure 4-33 First Scaled accelerograms for 8-storey building located in Vancouver Class E site	102
Figure 4-34 Second calibration to reduce the deviation of mean spectrum less than 10% with target spectrum for Vancouver 8-storey Class E site.	102
Figure 4-35 Period range of interest for 3-and-8 storey buildings site class C.	104
Figure 4-36 Scaling the ground motions within the TRS1 range for site class C.	104
Figure 4-37 Scaling the ground motions within the TRS2 range for site class C.	106
Figure 4-38 Period range of interest for 3-and 8-storey buildings site class E.	107
Figure 4-39 Scaling the ground motions within the TRS1 range for site class E.	107
Figure 4-40 Scaling the ground motions within the TRS2 range for site class E.	108
Figure 5-1 Inter-storey drift for Vancouver 3-storey site class C and E	112
Figure 5-2 Inter-storey drift for Vancouver 8-storey site class C and E	113
Figure 5-4 Comparison of the magnitude of overturning moment for 8-storey site class C and E	115
Figure 5-3 Comparison the magnitude of overturning moment for 3-storey site class C and E.	115
Figure 5-5 Inelastic rotation for 3-storey building in Vancouver on site class C and E	117
Figure 5-6 Inelastic rotation for 8-storey building in Vancouver on site class C and E	118

Figure 5-7 The ratio of V_{fmax}/V_{pr} for 3-storey structures on Site Class C and E.....	120
Figure 5-8 The ratio of V_{fmax}/V_{pr} for 8-storey structures on Site Class C and E.....	121
Figure 5-9 Normalized soil stress bearing for 3-storey building on site class C and E	122
Figure 5-10 Normalized soil stress bearing for 8-storey building on site class C	123
Figure 5-11 Normalized soil stress bearing for 8-storey building on site class E.....	123
Figure 5-12 Foundation settlement and uplift for a 3-storey building on site class C	124
Figure 5-13 Foundation settlement and uplift for a 3-storey building on Soil Class E site	125
Figure 5-14 Foundation settlement and uplift for an 8-storey building on site class C	126
Figure 5-15 Foundation settlement and uplift for an 8-storey building on site class E	126
Figure 5-16 Inter-storey drift for Montreal 3-storey frame, Class C and E sites	135
Figure 5-17 Inter-storey drift for Montreal 8-storey site class C and E.....	137
Figure 5-18 Comparison the magnitude of overturning moment for 3-storey on site class C and E	138
Figure 5-19 Comparison the magnitude of overturning moment for 8-storey on site class C and E	139
Figure 5-20 Inelastic rotations for 3-storey building in Montreal on site class C and E	140
Figure 5-21 Inelastic rotations for 8-storey building in Montreal on site class C and E	141
Figure 5-22 The ratio of V_{fmax}/V_{pr} for 3-storey structures on site class C and E	142
Figure 5-23 The ratio of V_{fmax}/V_{pr} for 8-storey structures on Site Class C and E.....	144
Figure 5-24 Foundation settlement and uplift for an 8-storey building on site class C	145
Figure 5-25 Foundation settlement and uplift for a 3-storey building on site class E	145
Figure 5-26 Foundation settlement and uplift for an 8-storey building on site class C	146
Figure 5-27 Foundation settlement and uplift for an 8-storey building on site class E	146
Figure 6-1 Vancouver 3-storey structure on soil class C STKO model.....	156
Figure 6-2 Vancouver 3-Storey structure on Soil Class C - A Sample for Moment Diagram....	157

Figure 6-3 Vancouver 3-Storey structure on Soil Class C - A Sample for shear force diagram	157
Figure 6-4 Vancouver 3-Storey structure on Soil Class C - A Sample for axial force diagram .	158
Figure 6-5 Vancouver 3-Storey structure on Soil Class C displacement.....	158
Figure 6-6 Vancouver 8-storey on soil class E STKO model	159
Figure 6-7 Vancouver 8-storey on site soil E.....	160
Figure 6-8 Vancouver 8-storey on site soil E.....	160
Figure 6-9 Vancouver 8-storey on site soil E.....	160
Figure 6-10 Vancouver 8-storey on site soil E.....	160
Figure 6-11 Sap2000 models for 3-storey structure soil class C and E	161
Figure 6-12 Sap2000 model for 8-storey structure soil class.....	161

LISTE OF SYMBOLS AND ABBREVIATIONS

A	Area
a_0	Dimensionless factor for radiation damping estimation
a_1, a_2, a_3	Steel02 material isotropic hardening parameter
ACI	American Concrete Institute
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) stiffness.
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.
EBF	Eccentric braced frame
c_0, c_1	Rayleigh damping coefficients
CM	Center of mass
CP	“Capacity protected” foundation type
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
D/C	Demand / capacity ratio
DAA	Direct analyses approach for soil-structure interaction implementation
DBE	Displacement-based element
DCE	Deformation controlled elements
DOF	Degree of freedom
DS	Ductile segment
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
ex	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 of 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 inertial about y axis.
J	Torsional constant or 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}	Rotational stiffness of the footing about the y axis
L	Length of a given element; live load
l_f	Length of the footing (perpendicular to the axis of rotation).
L1, L2, L3	Thornton imaginary columns length
L_{ave}	Average of the length L1, L2 and L3
L_{end}	Length of the stiffer end region a footing needed to reflect rotational stiffness
L_f	Overall length of the footing

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
NCF	Non-capacity-factored $R_dR_o=6$
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 sub element
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_{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	The ratio between the spring's stiffness at the end and in the middle of the footing region
R_o	Overstrength
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
SSP	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_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_{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_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
α_s	Length of uniform bearing stress in soil of rock required to resist the applied loads
α_z	Stiffness modification factor for radiation damping estimation
β	Damping
β_f	Foundation damping
β_i	Assumed structural damping with fixed base
β_o	Flexible base system damping
β_z	Vertical radial damping
Δ_r	Roof displacement due to foundation rotation
Δ_u	Structural maximal displacement having an inelastic response
Δ_w	Roof displacement with fixed base
Δ_y	Structural displacement at yielding resistance
δ_{ave}	Average of the displacements at the extreme point of the structure at level produced by the applied lateral force F_x
δ_{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 $\pm 0.10D_n x$ from the centres of mass (CM) at each floor diaphragm

γ_s	Density of the soil or rock
θ	Footing rotation in radians
η_z	Modification factor of rotational stiffness about y axis due to the embedment effect of the footing
μ	Ductility demand
v_s	Average effective shear wave velocity in site soil conditions
ϕ	Resistance factor
ψ	Dimensionless factor for radial damping estimation
Ω_0	Overstrength factor a a given SFRS according with ASCE7-16
ω	Natural frequency

LIST OF APPENDICES

APPENDIX A STKO MODELS AND DIAGRAMS	156
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CHAPTER 1 INTRODUCTION

1.1 Context

Canada is an earthquake-prone country, and all structures have to be designed for seismic loads. A common design method employs the capacity design. The capacity design approach acknowledges that not all elements within a structure have identical strength or ability to support loads. Certain sections of the structure are intentionally designed to be weaker but more ductile than others. This deliberate diminishing of the resistance of specific elements while making them ductile elements enables them to dissipate energy and experience-controlled permanent deformation during severe seismic events, while safeguarding the more critical components from damage.

In capacity design, foundations of building structures are considered to be capacity protected elements and as such are designed to develop the capacity of seismic force resisting systems (SFRS). Therefore, an elastic response of these elements is envisioned under seismic loads. For unanchored foundations, Canadian provisions recognize that at a certain load level the foundation will “rock”, thereby limiting the seismic demand on the superstructure. However, the yielding of the superstructure remains the primary mechanism of energy dissipation. This approach to foundation design aims in principle to avoid energy dissipation in the foundations and the underlying soil and can be justified in view of the difficulty to detect post-earthquake damage in foundations and the high cost of possible repairs.

The impact of a rocking foundation on capacity design for steel frame structures, especially in earthquake-prone regions, may be important. Rocking can provide an alternative energy dissipation mechanism during earthquakes and reduce the ductility demand on the superstructure. Redistribution of seismic forces occurs, modifying dynamic response of the super structure. On the other hand, rocking of foundation increases inter-storey drifts because of foundation rotations which necessitates consideration in capacity design of the superstructure.

Steel eccentrically braced frames are popular steel seismic load resisting systems in buildings, offering a good combination of resistance, rigidity and ductility. The critical components of EBFs are the ductile links, formed within the beams between their intersection with bracing members. They are intended to yield, either in shear or in bending, and undergo inelastic deformations under applied lateral forces, dissipating in this way a significant portion of the seismic energy in the

process. This energy dissipation realised in links, limits the magnitude of forces transferred to other sections of the structure, thus reducing the risk of damage or collapse.

The seismic design requirements for EBFs have been integrated into Canadian and American design standards, among others, since the 1980s. Canadian steel design standard, CSA-S16 standard employs the capacity design approach, aiming to size the link element for the seismic forces specified by the National Building Code of Canada (NBCC), while keeping the other frame members protected. To do so, the design forces for other members of the frame are determined based on the link's strength, taking into account its yielding, plastic deformation, and strain hardening.

The requirements for seismic design of foundations, as outlined in the Canadian steel design standard, relate to the Canadian concrete design standard, CSA A23.3. The research backing up foundation design requirements has mostly been done on research conducted on concrete shear walls and does not fully reflect the reality of seismic behaviour of steel braced frames.

While in reinforced concrete walls the concentrated plasticity develops at the base of the wall, in braced steel frames, ductility is distributed and requires that dissipative mechanisms be present in all shear links of the frame. Consequently, as foundations are designed as protected elements, this may result in a large design load on the foundation. This situation can potentially lead to overly conservative foundation design and excessive estimates of overall building drift, induced by foundation rotations. The impact is not limited only to the increase of foundations' dimensions and their cost but may also put in question the selection of steel as the material for the force-resisting system.

In Canada, two options are possible to design unanchored foundations: capacity-protected (CP) and not capacity-protected (NCP) options. CP foundations must be able to resist the factored gravity loading and the lateral load overturning capacity of SFRS. As mentioned above this is the preferred type of foundation as it is anticipated that the dissipation of seismic energy will take place primarily in the SFRS, and results in SFRS that has a more robust response to any unforeseen severe seismic demand, and additional deformation imposed to the superstructure due to foundation rotations are less important.

NCP foundations, previously called in Canadian codes “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 is still the intended source of energy dissipation and must be designed to develop the level of ductility anticipated for SFRS employed. According to A23.3-19, an NCP foundation must have an overturning resistance sufficient to resist factored gravity loads and the larger of: (i) 75% of nominal overturning capacity of SFRS and (ii) overturning moment resulting from the design load combinations including earthquake effects calculated using $R_d R_o = 2$. Nominal capacity is calculated considering resistance factors for concrete (ϕ_c) and steel reinforcement (ϕ_s) equal to 1, while R_d and R_o represent the ductility and overstrength factors, respectively, defined in NBCC for a chosen SFRS. In all cases, the maximum overturning capacity of foundations should not exceed the overturning moment caused by seismic loads calculated considering $R_d R_o = 1$.

Contrary to Canada, in the United States, capacity design approach is not mandatory for foundations. In fact, design practices vary significantly as a function of experience of design offices and client's requirements (Moehle 2015). Most often the foundations are designed for the same level of seismic loads as the ductile elements of the superstructure. This implies that some inelastic foundation response will occur at the level of design earthquake.

By creating full nonlinear models for each type of foundation that account for soil-structure interaction, the current study assesses various parameters. These parameters encompass the overturning moment generated at the foundation, the vertical displacements of the foundation, the axial forces in the springs simulating soil behaviour, the moment causing uplift of the footing, inter-storey drifts, and the shear force within the shear links. Subsequently, the results were compared with the demands stipulated by the Canadian codes.

Previous studies carried out on foundation design of concentrically braced frames (Reyes 2023) showed that some improvements of Canadian design procedure are possible, particularly for the competent soils. In this thesis, the study is extended to eccentrically braced frames in order to complete data on seismic design of the foundations of steel braced frames in Canada. Nonlinear time-history analysis, including an inelastic response of both steel braced frame and foundation-soil system is applied to investigate in detail the response of superstructure-foundation-soil system and validate current design procedures

1.2 Objectives

The main goal of this project is to validate the design criteria for shallow foundations of steel eccentrically braced frame based on the current standards in Canada and determine the more realistic behaviour of an eccentrically braced frame and its foundation by including the soil-structure interaction. More specifically this research project aims to:

- Assess seismic design provisions given in NBCC2020 and CSA-A 23.3-19 for the design of foundation for steel eccentrically braced frame systems.
- Determine if overturning design moments reflect seismic behaviour of EBFs.
- Explore possible improvements of design procedure.

1.3 Methodology

The methodology is summarized in the following steps:

- Conduct a literature review on the previous studies on seismic behaviour of EBFs, superstructure-foundation-soil interaction.
- Design two 3- and 8-storey buildings with EBFs located in moderate and high Canadian seismic zones (Montreal, Vancouver) on Class C and Class E sites.
- Design foundation according to Canadian design requirements (NBCC 2020, CSA S16-19 and CSA A23.3-19) considering Capacity Protected (CP) and Not-Capacity Protected (NCP) options.
- Design foundation without consideration of any capacity design requirements similar to the US design. This option will be referred to in the text as Non-Capacity Factored (NCF)
- Select and calibrate two sets of accelerograms for Vancouver and Montreal, compatible with NBCC 2020 design spectra.
- Develop a numerical model in STko/OpenSees that include soil-structure interaction and inelastic frame response
- Conduct a nonlinear time history analysis considering fixed-base conditions and soil-structure interaction.
- Assess the validity of Canadian design requirements for foundations of steel frames, using the results of NLTHA.
- Propose potential improvements of design procedures

1.4 Organization of the dissertation

This document is organized in 6 chapters and one Annex. 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 NLTHA and discussions are presented in the fifth chapter. Finally, in the chapter six the undertaken work is summarized and the conclusions and suggestions for further work are presented.

CHAPTER 2 LITERATURE REVIEW

This Chapter summarizes previous studies on seismic design and behaviour of steel eccentrically braced frames and the foundations with special consideration given to soil-structure interaction and it's divided into three sections. The first section reviews the seismic capacity design for superstructure and the shallow foundations as per the current Canadian codes and standards, namely the NBCC2020, the CSA A23.3-19 and the CSA S16-19. The second section treats the soil-structure interaction, and it's influence on seismic response of the superstructure. The third section presents STko/OpenSees software and reviews the numerical modeling of the soil, the foundation, and the elements of the superstructure for NLTHA.

2.1 Seismic design

The main objective of seismic design is to create structures that can effectively resist the forces and movements generated by earthquakes. The primary goal of seismic design is to protect human life by designing structures that can withstand earthquakes without causing loss of life due to collapse or structural failure. Seismic design also considers the functionality of structures after an earthquake. Designers aim to ensure that even if a structure undergoes damage, it can still perform its intended function and allow occupants to evacuate safely. Seismic design involves understanding the seismic hazard of a region, considering the characteristics of the local geology and ground motion, and then applying engineering principles to design structures that can effectively dissipate or absorb seismic energy. This is achieved through various techniques, including appropriate material selection, proper structural detailing, and adherence to building codes and regulations specific to seismic zones. For building structures in Canada three main design provisions namely NBCC 2020, CSA S16-19 and CSA A23.3-19 collaborate to establish a comprehensive seismic design approach for steel and reinforced concrete buildings. In NBCC 2020 seismic loads and analysis methods for use during seismic design are defined. As for the design procedures themselves, the CSA S16-19 standard directs the design of steel structures, while the CSA A23.3-19 standard assumes this role for concrete structures.

In this thesis, the superstructure being a steel braced frame, the design procedure follows the guidelines of CSA S16-19, while the concrete foundation is designed in accordance with CSA A23.3-19.

2.2 Superstructure

2.2.1 Loads and analysis methods

Loads

Design loads considered in seismic design are twosome: gravity and inertial loads. The vertical loads encompass the weight of the structure itself and any permanent or live loads it carries. The inertial loads represent the inertia forces induced by the structure's mass and its acceleration due to ground shaking. These forces are influenced by seismicity of design location and dynamic response of the structure in various modes of vibration. The seismic load cases to be considered in seismic design are:

- $1.0D + 1.0E$
- $1.0E+1.0D+0.5L+0.25S$
- $1.0E+1.0D+0.25L+0.5S$

The combination including only vertical loads that need to be checked are:

- $1.4D$
- $1.25D+1.5L+0.5S$
- $1.25D+0.5L+1.5S$

In these combinations, D, L, S, and E represent the dead, live, snow, and earthquake loads, respectively. Deformation limits are applied to horizontal inter-storey displacements based on the building's importance level. For civil protection buildings, the limit is $0.01h_s$; for schools, it's $0.02h_s$; and for all other buildings, it's $0.025h_s$.

P-Δ effects

The P-Δ effect in seismic design is a structural engineering phenomenon that accounts for the deformation-induced changes in internal forces and moments within a building or structure as it undergoes lateral displacement during an earthquake while being exposed to vertical loads.

In essence, the P-Δ effect recognizes that the response of a building or structure during an earthquake is not purely linear; it considers the feedback loop between lateral displacements and changes in internal forces. Neglecting the P-Δ effect can lead to inaccurate assessments of structural behaviour and potentially compromise the safety and stability of the structure during seismic

events. As presented in Figure 2-1, these effects result in a reduction in the lateral rigidity of the structure and must be considered in design of steel structures.

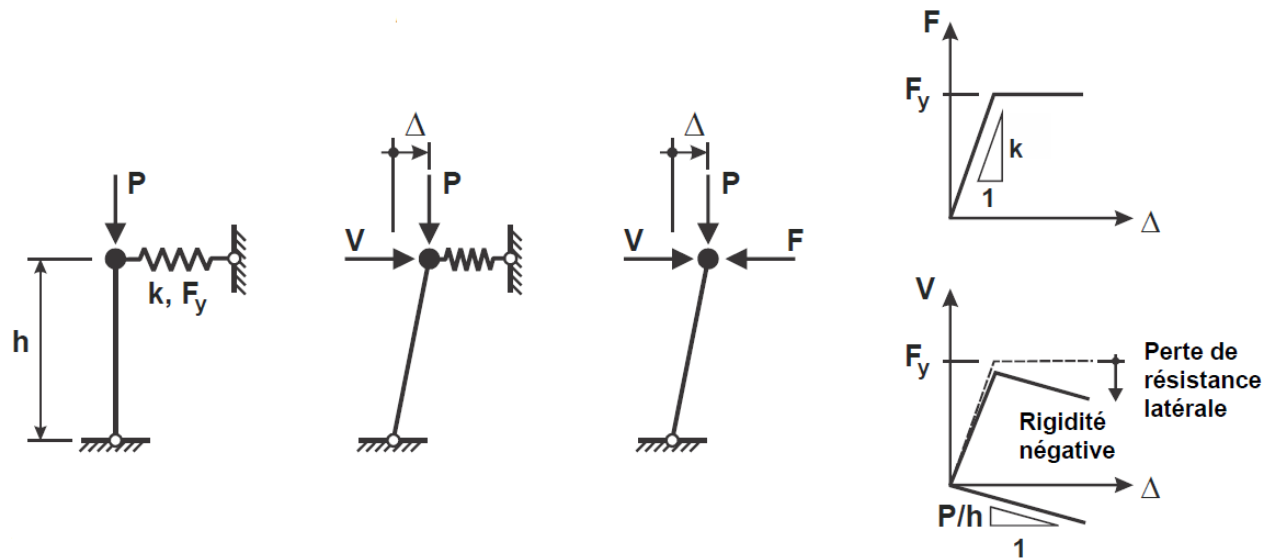


Figure 2-1: Decline of lateral resistance of the building caused by P-Δ effects (Tremblay 2021).

Equilibrium equation

$$\Sigma M = Vh + P\Delta - Fh = 0 \Rightarrow F = V + (P/h) \Delta \Rightarrow V = F - (P/h) \Delta$$

Equal to P-Δ effect the lateral resistance of the structure is reduced.

Analysis method

The National Building Code of Canada (NBCC) outlines two distinct techniques to determine design seismic forces for building structures and analyse their seismic behaviour: the equivalent static force procedure and dynamic analysis. The latter approach is favored due to its higher precision in accounting for the structure's dynamic characteristics, which significantly impact its seismic response. Dynamic analysis presents two options: response spectrum analysis and time history analysis. In the first approach, the design's peak deformation and force demand factors stem from a combination of responses in the primary modes of structural vibration. This method considers the linear elastic response of the structure, while the seismic input is defined by the NBCC's designated design spectrum. In contrast, the second approach solves equations of dynamic equilibrium at each time interval, thereby generating time-dependent seismic demand indicators

throughout the ground motion's duration. (Tremblay et al. 2015). These methods are presented in the following section.

2.2.1.1 Equivalent Static Force Procedure

The steps for Equivalent Static Force Procedure are:

- Identify the seismic zone where the structure is located. The seismic zone will determine the design parameters and forces to be applied.
- Determine the importance level of the structure (e.g., low, medium, high).
- Calculate Seismic Design Parameters T_a , $S(T_a)$, M_v , R_d , R_o and W
- Calculate the seismic base shear using the Equivalent Static Force Procedure (V_{ESF}) formula, which considers the mass of the structure, the design spectral response acceleration, and other relevant factors.
- Distribute the calculated seismic base shear along the height of the structure using a prescribed distribution pattern.

Equivalent Static Force is computed by this formula:

$$V_{ESF} = \frac{S(T_a)M_v I_E W}{R_d R_o} \quad (2.1)$$

Where:

- $S(T_a)$: spectral acceleration regarding to the fundamental period and target spectrum corresponding to a 2% probability of exceedance in 50 years.
- M_v : factor to account for higher mode effects on base shear.
- I_E : Earthquake importance factor of the structure
- W : Seismic weight of the building
- R_d : Ductility-related force modification factor reflecting the capability of a structure to dissipate energy through reversed cyclic inelastic behaviour (Figure 2-2).
- R_o : Overstrength-related force modification factor accounting for the dependable portion of reserve strength in a structure designed.

- T_a = The fundamental lateral period, according to NBCC2020 for the braced frames. It is equal to $0.025h_n$. However, as per the NBCC 2020 guidelines, it is permissible to employ a timeframe of up to twice the value of T_a if validated by an analysis using a numerical model.

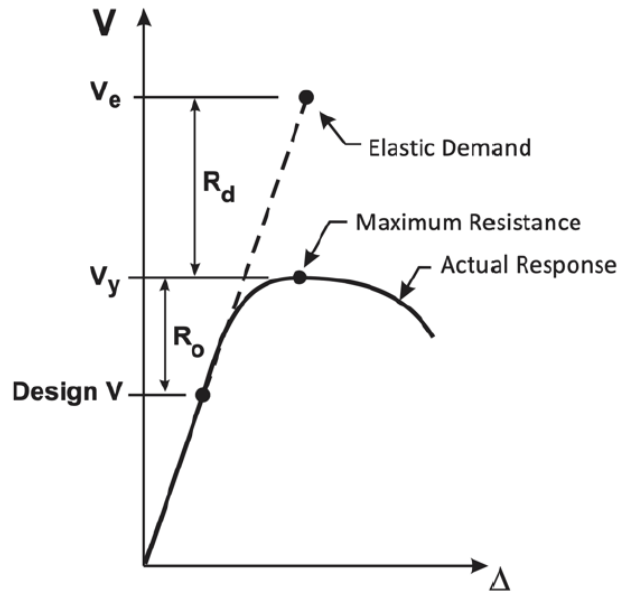


Figure 2-2 : Graphic representation of R_d and R_o factors (Tremblay 2021).

For the braced frames systems, V_{ESF} shall not be less than:

$$V_{min} = \frac{S(2.0)M_v I_E W}{R_d R_o} \quad (2.2)$$

For buildings having an SFRS with an R_d equal to or greater than 1.5, V need not be greater than the larger of:

$$V_{max} = \frac{\left(\frac{2}{3}\right) S(0.2) I_E W}{R_d R_o} \quad (2.3)$$

$$V_{max} = \frac{S(0.5) I_E W}{R_d R_o} \quad (2.4)$$

To determine $S(T_a)$, begin by calculating the period of the structure. Subsequently, extract $S(T_a)$ from the target spectrum with respect to site categories E or C and period T (see Figure 2-3 and Figure 2-4).

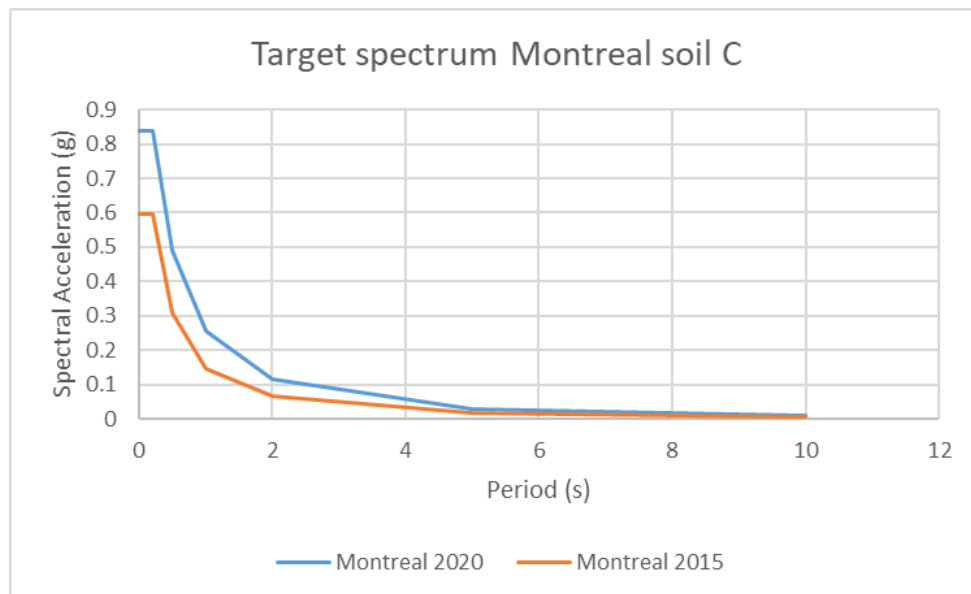


Figure 2-3 : The comparison of design spectrum for Montreal site category C increased between NBCC 2020 and NBCC 2015.

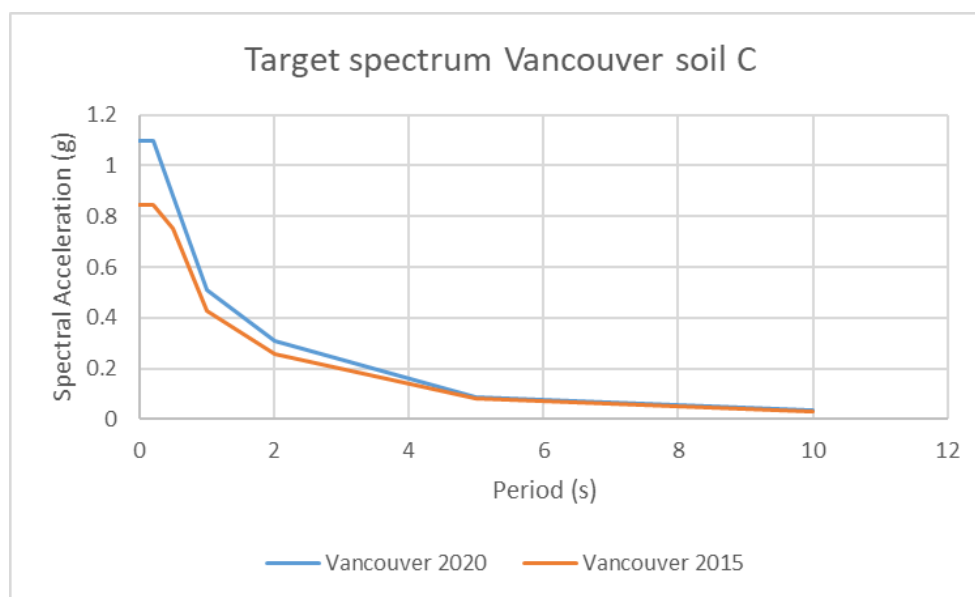


Figure 2-4 : The comparison of design spectrum for Vancouver site category C increased between NBCC 2020 and NBCC 2015.

Once the shear at the base is computed, this force should be distributed over the building's height which is illustrated in Figure 2-5. The lateral force at each floor (F_x) is established during the process of distributing the base shear, employing the following equation:

$$F_x = \frac{(V - F_t)W_x h_x}{\sum_{i=1}^n W_i h_i} \quad (2.5)$$

Where:

F_x = Lateral force on each floor

V = shear force at the base

F_t = Additional lateral force at the roof level

W = Seismic weight of the floors

h = Elevation of the floor from the ground

" F_t " denotes an abbreviation for an extra force applied at the top of the building to account for the influence of higher modes of vibration on the distribution of lateral forces.

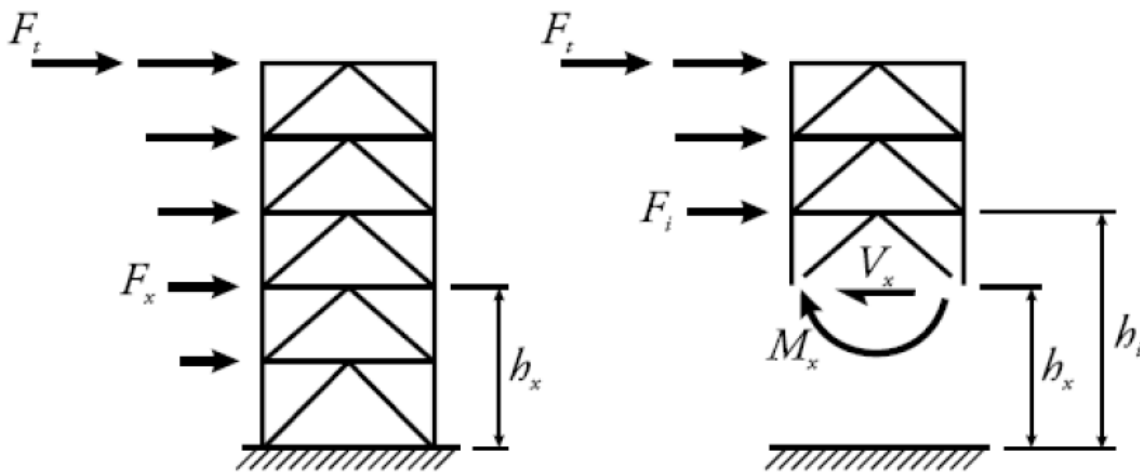


Figure 2-5 : Lateral static seismic forces distribution (Tremblay 2021).

Once the F_x was computed for each storey, the shear force and moment in the ductile link of eccentrically braced frames can be computed.

Moreover, the structure shall be designed to resist overturning effects caused by the earthquake forces F_x and the overturning moment at level x , M_x , shall be determined using the following equation:

$$M_x = J_x \sum_{i=x}^n F_i (h_i - h_x) \quad (2.6)$$

where

$$J_x = 1.0 \text{ for } h_x \geq 0.6h_n \quad (2.7)$$

$$J_x = J + (1 - J) \left(\frac{h_x}{0.6h_n} \right) \text{ for } h_x < 0.6h_n \quad (2.8)$$

2.2.1.2 Dynamic Analysis Procedure

The Dynamic Analysis Procedure is a method used in seismic design to assess a building or structure's response to earthquake forces. It is an advanced and comprehensive approach that considers the dynamic behaviour of a structure during an earthquake, including its natural frequencies, damping characteristics, and response to ground motion. Dynamic analysis is a more complex and sophisticated method when compared to static analysis, as it accounts for the time-varying nature of earthquake ground motion and the resulting dynamic forces on the structure.

In this method mathematical model of the structure is made, incorporating factors such as mass distribution, stiffness, and damping properties. Then the target spectrum that represent the expected seismic activity in the region is applied on the model. The analysis identifies the natural modes of vibration of the structure, which represent the different ways it can move in response to seismic forces. This information aids in understanding how the building will behave during an earthquake.

Furthermore, the procedure evaluates how the structure responds to the applied earthquake forces, taking into consideration factors such as displacement, acceleration, and inter-storey drift. This

thorough analysis helps ensure that the building can withstand the anticipated seismic forces without sustaining excessive damage.

According to NBCC 2020, the analysis for earthquake actions shall be carried out using the dynamic analysis except when the equivalent static force procedure is applicable. This is the case for all buildings located in low seismicity zones. For moderate and high seismic zones, equivalent static force procedure can be used for the regular structures that are less than 60 m in height and have a fundamental lateral period, T_a , less than 2 (s) in each of two orthogonal directions. It also can be used for the structures with a structural irregularity of Type 2, 3, 4, 5, 6 or 8 as defined in (Table 4.1.8.6 NBCC 2020) that are less than 20 (m) in height and have a fundamental lateral period, T_a , less than 0.5 s in each of two orthogonal directions.

The Dynamic Analysis Procedure shall be in accordance with one of the following methods:

- Linear Dynamic Analysis by either the Modal Response Spectrum Method or the Numerical Integration Linear Time History Method.
- Non-linear Dynamic Analysis, in which case a special study shall be performed.

The modal response spectrum method assumes that the linear elastic response of a structure can be approximated by summing the individual responses from each distinct vibration mode relevant to the structure. The overall maximum response is derived by combining the maximum values of these individual responses. The NBCC 2020 stipulates the utilization of a set of vibration modes that collectively account for at least 90% of the seismic mass in each direction of analysis.

To apply the modal response spectrum method, a detailed analytical model of the structure must be available, representing its components, connections, and interactions. This includes considering both the vertical and lateral load bearing elements. Then the elastic base shear (V_E) is computed according to the mass of the structure and spectral acceleration which gives the inertial loads. Multiplying V_E by the importance factor (I_E) and then dividing it by the ductility factor (R_d) and the overstrength factor (R_o) leads to the computation of the dynamic base shear (V_{DYN}). This base shear is then calibrated with respect to the base shear obtained by equivalent static force method. According to the NBCC guidelines, for regular structures, if the value of V_{DYN} is lower than V_{ESF} , a minimum force equal to 0.8 times V_{ESF} should be considered at the base. In cases of irregular structures under the same condition, V_{DYN} must at least be equal to V_{ESF} .

2.2.2 Capacity design

Unless otherwise specified in Clause 27 of CSA S16-19, seismic force-resisting systems shall be designed in accordance with capacity design principles. Capacity design, as outlined in CSA S16-19, is a seismic design approach aimed at ensuring the safety and integrity of steel structures during earthquake events. It involves meticulously controlling and distributing the strength and stiffness of various components within a structure to ensure that yielding and deformation predominantly occur in specific, controlled regions while maintaining the overall strength and stability of the structure.

Capacity design specifically focuses on achieving the required ductility (the ability to deform in plastic region without losing strength and stability). Naturally, the resistance and the rigidity of all elements of the building must be also granted. This involves the designation of particular components, such as steel braces or moment-resisting frames, to absorb seismic energy through plastic deformation while remaining stable. Engineers strategically designate certain elements as "weak links" within the structure, and detail them to be able to develop sufficient ductility. These designated elements are engineered to yield and dissipate seismic energy while safeguarding the more critical structural elements.

Furthermore, a good seismic design should minimize torsional effects in the structure, as these can lead to undesirable behaviour during an earthquake. Proper placement of lateral force-resisting systems and their connexion through diaphragms are essential to achieve this goal. Additionally, elements and connections in both the horizontal and vertical load paths are designed to withstand gravity and seismic loads, and diaphragms and collector elements are capable of transmitting the loads generated at each level to the vertical seismic force-resisting system. These loads are subsequently transmitted to the foundation.

Various types of ductile steel frames exist for capacity design. However, in line with the project's objective to validate foundation dimensions for steel eccentrically braced frames, the selected type is Type D (ductile) eccentrically braced frames, with $R_d = 4.0$ and $R_o = 1.5$ as per clause 27.7 of CSA S16-19. Ductile eccentrically braced frames dissipate energy by yielding of links.

2.2.3 Eccentrically braced frame

The primary concept in the development of an eccentrically braced frame (EBFs) Figure 2-6 is to combine the benefits of moment resisting frame (MRF) and a concentrically braced frames (CBF) for lateral load resistance within a single structural system. The EBFs system had its beginning during the 1970s with the objective of creating a structure that possesses both significant elastic stiffness and substantial energy dissipation capabilities during intense seismic events. Within EBF systems, yielding is specifically confined to the ductile links, while all other components of the frame are designed to maintain their predominantly elastic behaviour. Consequently, in the event of a severe earthquake, these links can be viewed as structural fuses that efficiently dissipate the seismic input energy through controlled and stable plastic deformations.

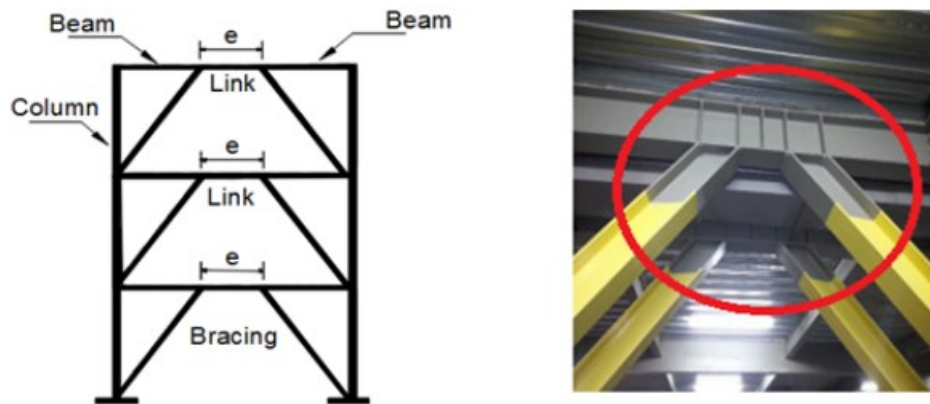


Figure 2-6 : Eccentrically braced frame (Nagui William 2019)

2.2.4 Link beam

2.2.4.1 Yield behaviour, shear capacity and overstrength

The dimension of a ductile link (e) is a crucial factor that governs the stiffness, strength, ductility, and performance of an EBFs. The link length ratio, denoted as ρ is calculated as follows, $\rho = e / (\frac{M_p}{V_p})$ where M_p represents the plastic moment capacity and V_p represents the plastic shear capacity of the link, offers a convenient way to assess the yield behaviour. The schematic representation of a ductile link is illustrated in Figure 2-8. When considering equilibrium, assuming equal end moments at the ultimate state, no interaction between moment and shear, and employing

an elastic-perfectly plastic material model, the theoretical threshold for the link length ratio that distinguishes shear-dominated from flexure-dominated behaviour is $\rho_{\text{theoretical}} = 2.0$.

In brief, for short (or shear) links, the primary mode of yielding involves shear deformation of the web, as depicted in Figure 2-7 (a). Conversely, in long (or moment) links, flexural yielding governs the link's behaviour, as shown in Figure 2-7 (c). An intermediate link, however, exhibits a combination of both shear and flexural yielding, as illustrated in Figure 2-7 (b).

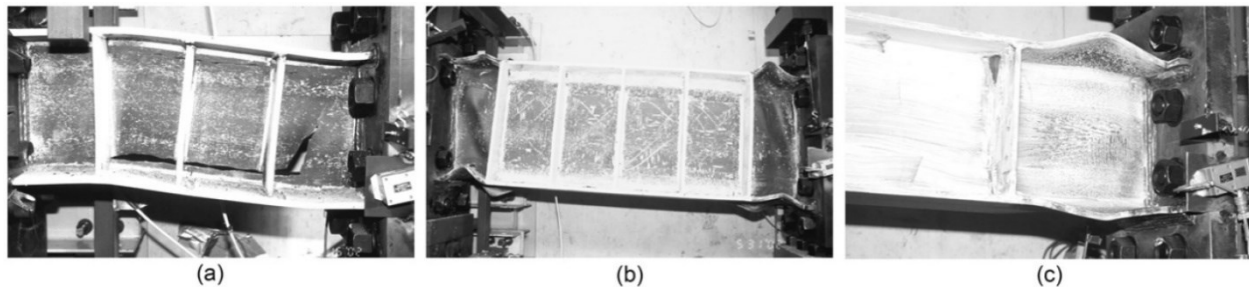


Figure 2-7 : Failure mechanisms of (a) short, (b) intermediate, and (c) long links (T. Okazaki 2005).

In the initial experimental investigations conducted by Roeder (1977) as well as Hjelmstad (1983), it was observed that short links exhibit significantly superior performance compared to long links when subjected to intense cyclic loads. This enhanced performance was noted in terms of both strength and ductility. Over the course of several years, Popov and his associates proposed various practical thresholds for achieving shear-dominated behaviour, ultimately settling on a limit of ρ being less than or equal to 1.6. The initial comprehensive examination of the performance of intermediate and long links (with ρ greater than or equal to 1.6) was carried out by Engelhardt et al. (1989). They conducted a series of 14 tests on 12 specimens at two-thirds scale, covering a range of ρ values from 1.45 to 4.25. The experimental findings led to the conclusion that there is a gradual shift from shear-dominated behaviour to flexure-dominated behaviour as ρ increases from 1.6 to 3. Despite this observation, in most of the past and current design provisions, links with length ratios falling within the range of 1.6 to 2.6 are categorized as intermediate links, while links with ρ greater than or equal to 2.6 are generally referred to as long links.

In brief, the ductile link beam designed to yield, either in flexure or in shear, prior to yielding of other parts of the eccentrically braced frame. Yield in shear force is often preferred for the following reasons:

- When link is shorter ($e < 1.6 M_p/V_p$), it results in greater lateral stiffness of the braces.
- Steel yielding in shear is more robust, which gives a more stable behaviour and a greater ability to dissipate energy.
- Bending moments imposed on the beam outside the link (and to the diagonals) are weaker because they remain less than M_p .

2.2.4.2 Link length limits

The link beam shall be either a segment of the beam or a modular link distinct from the rest of the beam. In this study the link beam is a segment of the beam. Link beams shall be Class 1 and designed for the coexisting shears, bending moments, and axial forces. Link beams may have Class 2 flanges and Class 1 webs when $e \leq 1.6 M_p/V_p$, where e is the length of the link and $V_p = 0.55 w_d F_y$.

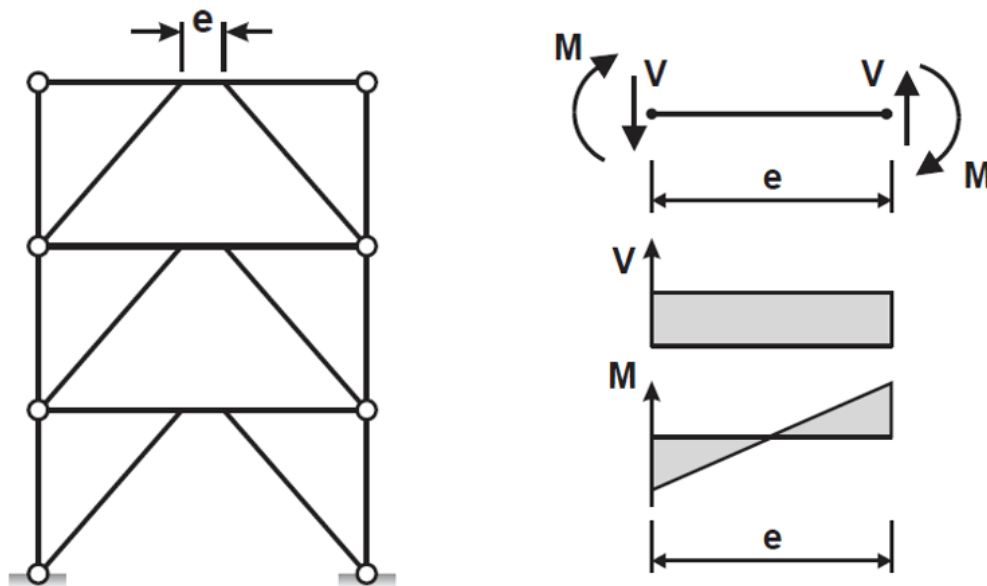


Figure 2-8 : Ductile segment (Tremblay 2021).

According to Figure 2-8 $M = V * e/2$, If the strengths of the link in bending and shear are M_p and V_p respectively, yielding of the link will take place when $V = 2M_p / e$ (in bending) and $V = V_p$ (in shear). Therefore, shear yielding will happen if $V_p < 2M_p/e$, or when e is less than $2M_p/V_p$. The theoretical limit $e = 2 M_p/V_p$; however, in reality, yielding is anticipated to occur in shear only when e is less than $1.6 M_p/V_p$, and in bending only if e is greater than $2.6 M_p/V_p$. The factored shear resistance of the link shall be taken as the lesser of $\phi V'_p$ and $2\phi M'_p/e$.

$$V'_p = V_p \sqrt{1 - \left(\frac{P_f}{AF_y} \right)^2} \quad (2.9)$$

$V_p = 0.55wdF_y$ for links with wide-flange cross-sections

P_f = axial force in the link

A = gross area of the link beam

$$M'_p = 1.18M_p \left[1 - \frac{P_f}{AF_y} \right] \leq M_p \quad (2.10)$$

2.2.4.3 Link rotation demand and capacity

The shear rotation angle of a link denoted as " γ " is the measurement of the angle between the link and the beam outside the link. Within this total rotation angle, there exists an inelastic portion, referred to as the "inelastic (or plastic) link rotation angle," represented by " γ_p ." In broad terms, the inelastic rotation capacity of a link, labeled as " γ_{pmax} " is described as the maximum inelastic rotation angle achieved during a cyclic test. The elastic portion of link rotation is relatively small, typically less than 0.01 radians.

To establish the inelastic rotation capacity, the link is subjected to a cyclic test involving at least one full loading cycle. The test continues until the shear resistance provided by the link drops below a predefined threshold. This threshold can be specified in three ways: (i) 80% of the highest recorded shear for the link during the cyclic test ($0.8V_{max}$), (ii) the plastic shear capacity (V_p), and (iii) the shear strength specified by engineering codes (V_n) for the link. Figure 2-9 serves as an illustrative example demonstrating the application of the first definition for assessing the inelastic rotation capacity, denoted as " γ_{pmax} " in a specimen tested by Okazaki et al. (2005).

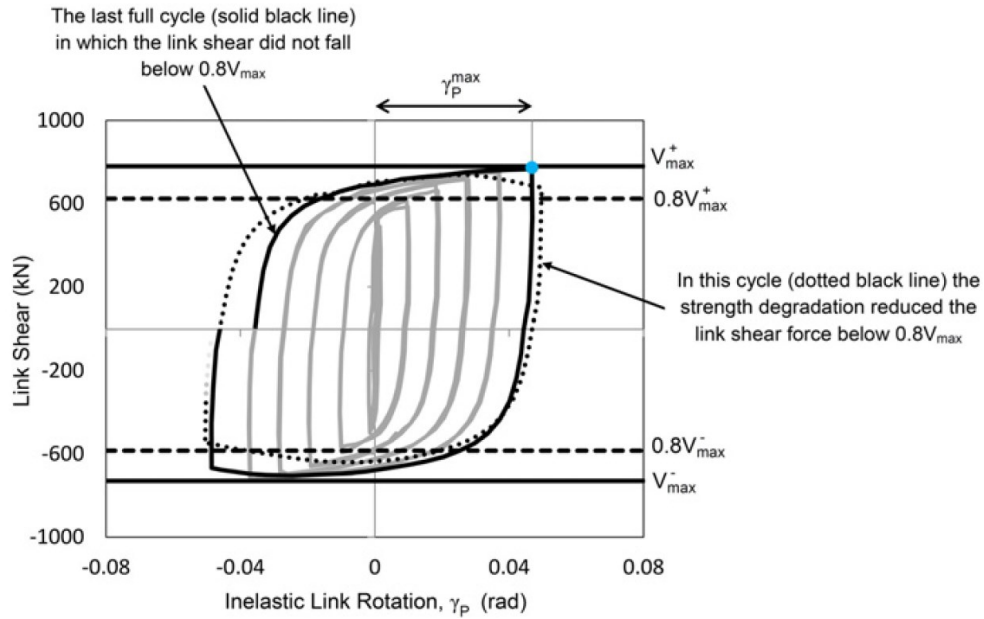


Figure 2-9 :Inelastic link rotation capacity for the UTA 9 specimen (Okazaki, T. 2005)

Empirical investigations have revealed that the capacity of links to rotate is significantly influenced by several factors, including the ratio of link length, the loading history, the compactness, and the rigidity of the web.

The plastic rotation in the ductile link is determined based on the estimated inter-storey displacement. In this calculation, Θ_p equals three times the inter-storey displacement under the design seismic load, without multiplication by $R_d R_o$.

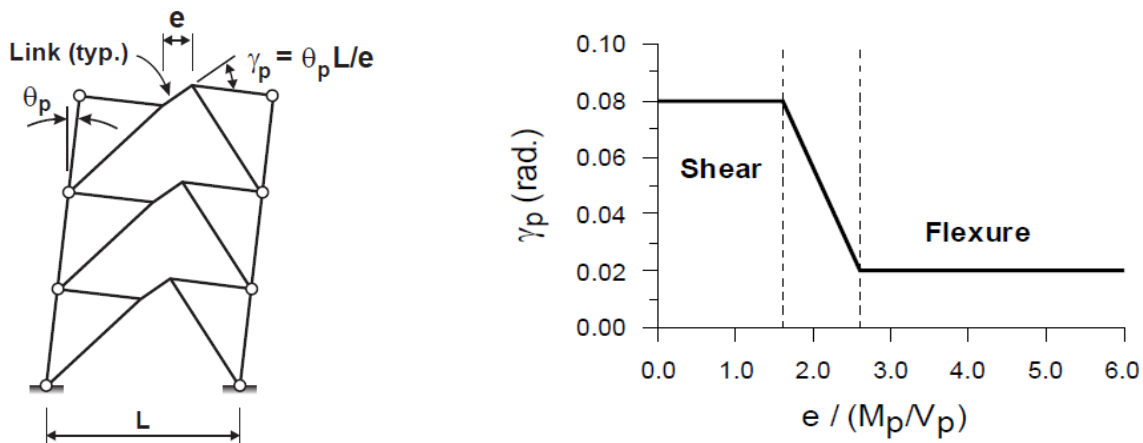


Figure 2-10 : The plastic rotation in DS changes according to the length of the link. (Tremblay 2021).

The inelastic link rotation shall not exceed the following limits:

- 0.08 radians when $e \leq 1.6 M_p/V_p$
- 0.02 radians when $e \geq 2.6 M_p/V_p$
- When $1.6 M_p/V_p < e < 2.6 M_p/V_p$ linear interpolation may be used.

2.2.4.4 The link overstrength

The nominal shear capacity (V_n) of a link can be defined as follows:

$$V_n = \text{Min} \left[V_p, \frac{2M_p}{e} \right] \quad (2.11)$$

The plastic capacity, specifically V_p and M_p , are determined by using the nominal yield stress, F_y , as the basis for calculation. When the length ratios of links are under 2.0, the first term takes precedence. However, for longer links, the second term becomes the dominant factor. To achieve the goals of the capacity design approach, it is essential to calculate the highest shear force that a link can sustain during a severe loading, denoted as V_{max} . To obtain a reliable estimation of V_{max} , the link overstrength concept is typically applied as follows:

$$V_{max} = \Omega (R_y V_n) \quad (2.12)$$

Here, R_y represents the ratio of the expected yield stress to the nominal yield stress, which is determined for each steel grade through statistical data while Ω , is the overstrength factor resulting from strain hardening. Popov et al. (1984) experimental research proposed a value of $\Omega = 1.5$ for design purposes, and this value continues to be utilized in most seismic design provisions.

The link overstrength values identified in earlier experimental studies by, Hjelmstad (1983), Kasai (1986), Engelhardt (1989), X. Ji (2015), concerning horizontal I-shaped links are displayed in Figure 4. Additionally, the conventional line representing $\Omega = 1.5$ is shown in this figure. (Sina Kazemzadeh Azad, 2017).

Usually, the nominal shear capacity of short links is computed by primarily considering the web area without taking into account the contribution of the flanges (AISC, 2010). Nevertheless, as pointed out by McDaniel et al. (2003), in cases of short links with relatively thick flanges, a substantial shear force is effectively supported by the flanges. To address this, Manheim and Popov

(1983), as well as Richards (2004), have introduced methodologies for assessing the plastic shear capacity of the link while accounting for the impact of the flanges. Richards (2004) conducted nonlinear finite element analysis on isolated links and showed that an overstrength factor of 1.5 is a suitable limit, even for very short links, as long as the flange effect is taken into consideration when determining V_p . In 2007, Okazaki and Engelhardt presented the findings from tests conducted on 37 links made from ASTM A992 steel. While these links were not considered very short, some of the specimens had notably high ratios of flange to web area (A_f/A_w). Surprisingly, these particular specimens did not demonstrate overstrength values significantly exceeding 1.5. Conversely, recent results from Ji et al. (2015), suggested that even very short links with lower ratios of flange to web area could achieve remarkably high overstrengths, occasionally surpassing 2 (Sina Kazemzadeh Azad, 2017). Moreover, for the links yielding mainly in bending, this excess resistance is lower and equal to around 20% of V_p .

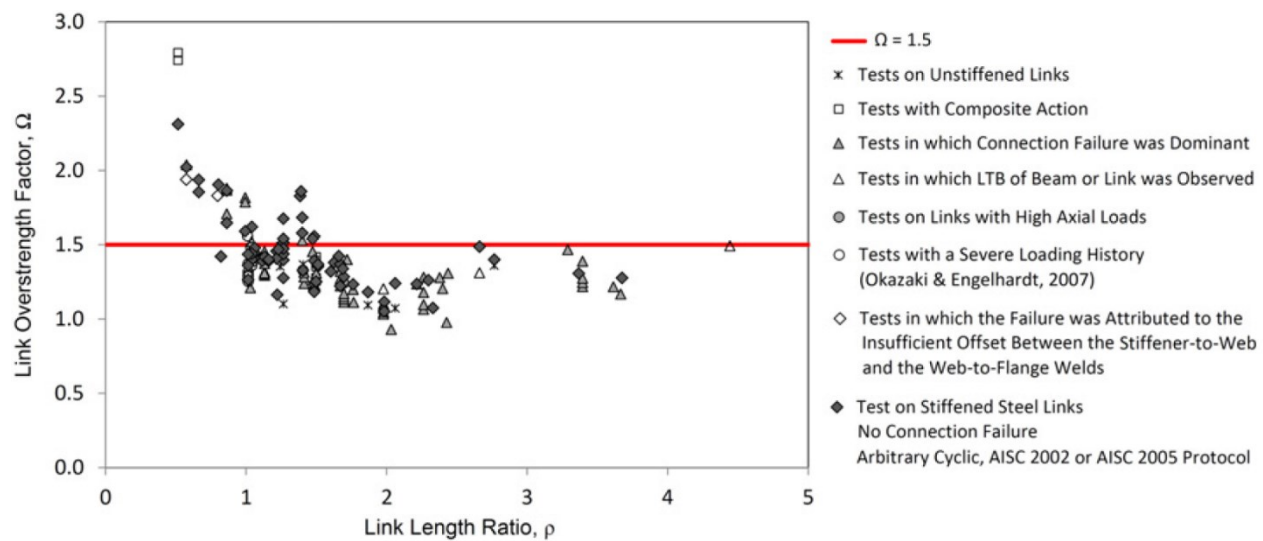


Figure 2-11 : Link overstrength factors reported in different experimental studies. (Sina Kazemzadeh Azad, 2017).

The numerical findings led to the conclusion that the inclusion of axial restraints can notably enhance overstrength, with reports of increases of up to 15% in certain models, particularly for short links. In a more recent study, Ji et al. (2015), also noted that constraining axial deformations of very short links through adjacent members can generate substantial axial forces within these links and have a considerable impact on their behaviour.

Another potential explanation for the remarkably high overstrength observed in certain tests is the significant cyclic hardening of steel caused by extensive plastic strains. Ji et al. (2015), introduced this concept based on an experiment by Kasai et al. (2004), involving robust, short steel panels where hardening persisted even at very large shear angles. Based on this observation Ji et al. (2015), proposed that the exceptionally high overstrength seen in very short links may be explained by their substantial rotational capacities. These links undergo significant plastic rotations, often exceeding 0.08 radians, which means that their webs are subjected to pronounced shear strains. This can result in excessive cyclic hardening. A more detailed finite element analysis conducted by Ji et al. 2015, on very short links demonstrated that for a link plastic rotation on the order of 0.15 radians, the cyclic hardening effect can enhance shear strength by approximately 70%.

2.2.4.5 Elements outside of the link

Most of the research on EBFs focused on the shear link behaviour, and it was assumed that all the other frame components remain elastic during the earthquake. An examination of the non-linear dynamic responses of multiple EBFS designed for diverse Canadian regions, conducted by Koboevic and Redwood (1997) has uncovered potential shortcomings in existing design methodologies. In regions with significant seismic activity, the link's shear forces exceeded the design expectations, resulting in yielding in the outer segments of the beams, significant overloading of columns, and, to a limited extent, braces in the upper levels of the structure. The magnitudes of link shear forces, deformations, and column forces in the uppermost storeys were significantly greater than design predictions. Furthermore, the columns were initially designed solely to withstand axial forces, without considering the bending moments that may arise during an earthquake due to the continuity of these columns over multiple floors. Kasai and Han (1997) recommend allocating fifteen percent of the column's resistance to account for these end moments. In a similar vein, Koboevic (2000) suggests increasing this allocation to forty percent for the upper section of the column. Consequently, the current Canadian standard (CSA-S16-19) mandates higher link over-strength factors and reserves a portion of the column's resistance to address this moment demand.

Creating a moment-resisting connection between the beam and the brace is a practical approach to decreasing the beam's cross-section. This connection enables the transfer of a portion of the moment

generated by the link to the brace, thus alleviating stress on the beam. Generally, a rigid connection shifts approximately 5 to 20% of the moment towards the brace (Koboevic and Redwood, 1997), and a more robust brace component enhances this ratio. This distribution can be estimated by assuming elastic behaviour for these two elements. It's important to note that the value of M_{beam} is restricted to the plastic capacity of the beam. Based on these assumptions, the moments sustained by these members can be calculated using the equations provided by Han et al. (1997).

$$M_{beam} = \min(M_{beam\ trans}, M'_p) \quad (2.13)$$

$$M_{beam\ trans} = \left[\frac{3r}{\psi \cos \theta + 3r} \right] M_f \quad (2.14)$$

$$M'_p = \left[1 - \frac{T_f}{AF_y} \right] M_p, \text{ in tension} \quad (2.15)$$

$$M'_p = 1.18M_p \left[1 - \frac{C_f}{AF_y} \right], \text{ in compression} \quad (2.16)$$

$$M_{brace} = M_f - M_{beam\ trans} \quad (2.17)$$

Where:

A = cross-sectional area of the beam

M_f = moment coming from the link, including over-resistance

M_p = plastic moment of the beam around its strong axis ($=Z_x F_y$)

r = ratio between the moment of inertia of the beam and the moment of inertia of the brace (bending in the plane of the frame)

T_f = factored tensile force in the beam outside the link generated by the link, including the over-strength

Θ = angle of the diagonal

Ψ = 3 if the connection at the bottom of the diagonal is hinge or 4 if rigid

Canadian code requirement for element outside of the link

According to CSA S16-19 the column sections should withstand $1.15 R_y$, times the nominal resistance of the ductile segments, except for the two upper floors where the specified extra strength requirement is $1.30 R_y$. The lower coefficient equal to 1.15 for the lower floor column calculations reflects the lower likelihood that all links will simultaneously reach their maximum overstrength. The axial loads resulting from this extra strength should not exceed 65% of the column capacity on the last two levels and 85% for the other columns. This restriction on column strength indirectly accounts for the bending moments that occur during yielding of ductile link in an earthquake. Columns shall satisfy the Class 1 or 2 limits in Table 2, CSA S16-19. Columns shall be designed to resist the cumulative effect of yielding links together with the gravity loads. The link forces shall be taken as the probable resistances of the links except in storeys below the top two storeys, where the link forces may be taken as 0.90 times the probable resistance of the links. Columns in multi-storey buildings shall be continuous and of constant cross-section over a minimum of two storeys; and satisfy the requirements of Clause 13.8, CSA S16-19 assuming an additional bending moment in the direction of the braced bay of $0.2ZF_y$ in combination with the computed bending moments and axial loads. In the top two storeys, the additional bending moment shall be taken as equal to $0.4ZF_y$.

The beam outside the link shall resist forces corresponding to the probable resistance of the link (Clause 27.7.3.2, CSA S16-19). When subject to these forces, the beam resistance may be determined using $R_y F_y$ and $\phi = 1.0$ when the link and the beam outside the link are part of the same beam and it shall be Class 1 or 2. According to CSA S16-19 the probable resistance of shear link is equal to $V_{\text{prob}} = 1.30 R_y V_p$. Factors 1.3 mainly represent the effect of strain hardening. For W profiles, $R_y = 1.1$, but $R_y F_y > 385 \text{ MPa}$. For columns, the probability that all ductile links in the levels above the storey under consideration simultaneously reach their probable resistance decreases with the number of levels. The factors 1.3 above can be multiplied by 0.9 for the links in the floors below the two upper floors.

Brace sections shall satisfy the Class 2 limits. Each diagonal brace and its end connections shall have a factored resistance to support axial force and moment produced by the link developing its probable resistance.

The connections must be able to resist the forces corresponding to the probable resistances of the link beams.

2.3 Foundation

The foundation serves as the component responsible for transmitting forces between the soil and the structure. Consequently, both the soil's properties and the loads transmitted by the structure will impact the foundation's size. These loads may encompass a combination of gravity loads, seismic forces, or wind loads. In the case of seismic and wind loads, they are not symmetric when transferred to the foundation via columns, potentially leading to an overturning moment. In calculations, this overturning moment is treated as an off-centre vertical force positioned in a manner that achieves an equivalent effect on both the foundation and the soil.

2.3.1 Foundation provisions according to NBCC 2020

Several considerations must be taken into account for foundation design. A foundation should be able to transfer various loads, such as gravity and inertial loads, to the soil surface. These loads are transmitted through the superstructure and then transferred by columns to the foundation. Inertial loads, like those caused by earthquakes and wind, result in overturning moments. In design, there are different approaches to determine soil stresses under foundation induced by the combination of vertical forces and overturning moments. Regardless of the assumed distribution of soil stresses, it is always considered that a part of the foundation under tension does not resist force and the overturning moment is balanced by the vertical force positioned eccentrically as illustrated in Figure 2-12.

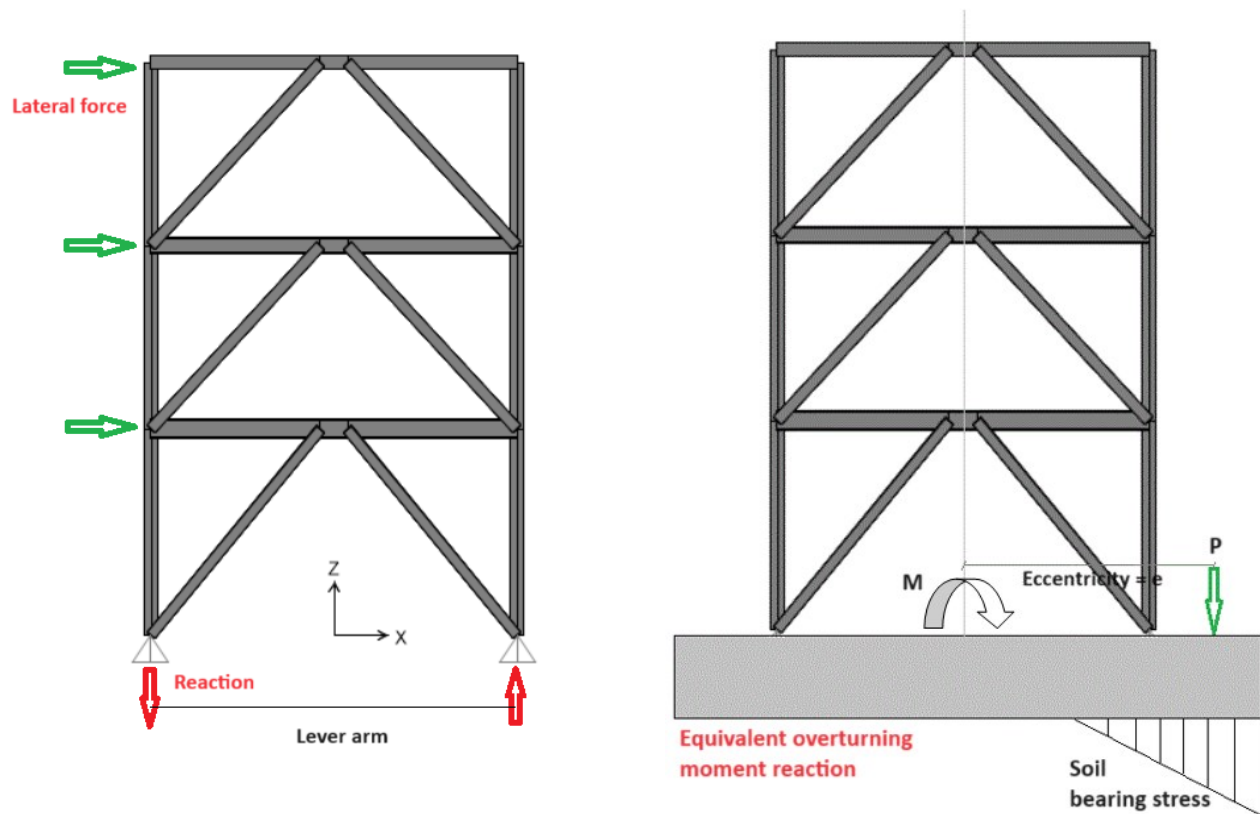


Figure 2-12 : Equivalent overturning moment reaction

Eccentricity, e , can be estimated by following expression:

$$e = \frac{M}{P} \quad (2.18)$$

M = overturning moment

P = Vertical load which takes into account the weight of both the soil and the foundation.

In the foundation design process, it is necessary to confirm four primary design criteria to guarantee the correct response to both vertical and lateral loads: (i) overall stability, (ii) soil load-bearing capacity, (iii) soil settlement, and (iv) foundation resistance.

Stability is realized when the eccentric force remains within the foundation's area. Thus, in the initial stage of the design process, an initial footing dimension is chosen and adjusted until stability is achieved.

Following that, an assessment is conducted to ensure that the maximum soil stresses ($q_{\max}$) remain below the factored bearing capacity of the soil (q_{fact}). A resistance factor denoted as " ϕ " set at 0.5,

is utilized to estimate the factored bearing resistance based on the soil's ultimate bearing capacity (q_u).

$$q_{max} \leq q_{fact} \quad (2.19)$$

$$q_{fact} = \phi q_u \quad (2.20)$$

The estimation of stress distribution beneath the foundation (q_{max}) can be achieved through the utilization of elastic techniques, until the plastification of the soil occurs. Figure 2.22 illustrates the variation in soil stress distribution for elastic and inelastic stress distributions. The equations for calculating q_{max} in each scenario are provided within the same figure.

When the condition $0 < e < h/6$ is met, the elastic stress distribution is determined as follows:

$$q_{max} = \frac{P}{A} \pm \frac{M_x}{S_x} \quad (2.21)$$

Where:

P = Vertical load.

A = Contact area between the footing and the soil.

M_x = Overturning moment.

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

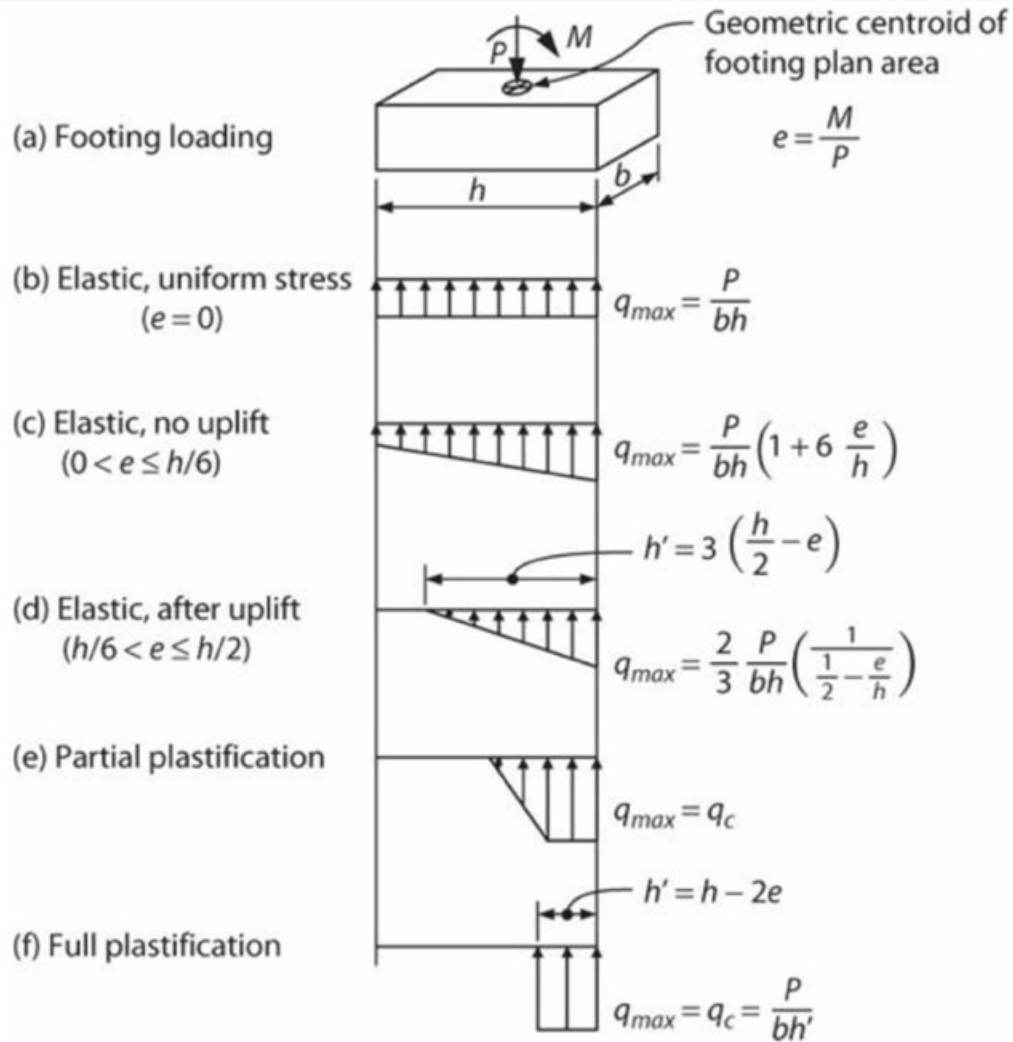


Figure 2-13 : Stress distribution under shallow foundation. (Moehle 2015)

Requirements relevant to foundation design, given in the clause 4.1.8.16 in NBCC 2020 can be summarized as follows:

- The increased displacements of the structure resulting from foundation movement shall be shown to be within acceptable limits for both the SFRS and the structural framing elements not considered to be part of the SFRS.
- Foundations shall be designed to have factored shear and overturning resistances greater than the lateral load capacity of the SFRS.

- The shear and overturning resistances of the foundation determined using a bearing stress equal to 1.5 times the factored bearing strength of the soil or rock and all other resistances equal to 1.3 times the factored resistances need not exceed the forces determined in Sentence 4.1.8.7.(1) NBCC 2020 using $R_d R_o = 1.0$, except that the factor of 1.3 shall not apply to the portion of the resistance to uplift or overturning resulting from gravity loads.
- A foundation can possess a factored overturning resistance lower than the lateral load overturning capacity of the supported Seismic Force-Resisting System (SFRS), under the condition that neither the foundation nor the supported SFRS is restricted from rotating. Additionally, the design overturning moment of the foundation should not be below 75% of the overturning capacity of the supported SFRS, and it should not be lower than the value calculated in Sentence 4.1.8.7.(1) using the factor $R_d R_o = 2.0$.
- The design of foundations shall be such that they are capable of transferring earthquake loads and effects between the building and the ground without exceeding the capacities of the soil and rock.

2.3.2 Shallow foundation

Shallow foundations are typically placed within the top layers of the soil, usually at a depth where the bearing capacity of the soil is sufficient to support the applied loads. The depth is relatively small compared to deep foundations like piles or caissons. Additionally, in this type of foundation, the depth is smaller than the width as illustrated in Figure 2-14. The NBCC2020 recognizes several types of shallow foundations, including isolated footings, strip footings, combined footings and mat or raft foundations. Shallow foundations work by spreading the building loads over a larger area of soil to prevent excessive settlement and must be designed to ensure that the bearing capacity of the soil is not exceeded. This involves considering factors such as the type of soil, its strength, and its settlement characteristics.

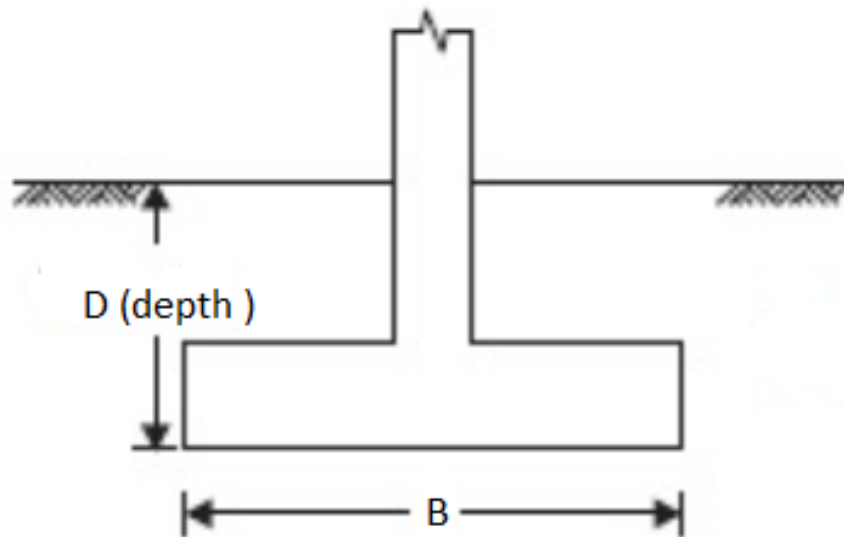


Figure 2-14 : The depth of a shallow foundation is less than its width $D < B$.

2.3.3 Rocking foundation

The notion of permitting foundations to "rock" as a strategy to reduce seismic pressures is widely recognized. This approach enables Canadian designers to design foundations for stress levels that are as less as 50% of the demand seen in linear-elastic conditions for new constructions (Adebar P, 2015). However, foundation rotations increase the displacements of the superstructure which in turn must be considered in design.

Significant changes have been implemented in the foundation design provisions of the 2015 editions of NBCC and they remain unchanged in NBCC2020. Based on these modifications, foundations are now distinctly classified as either constrained against rotations or permitted to rotate freely. In the latter scenario, the amplified horizontal deflections and drifts must be taken into consideration during the design of seismic force resisting systems (SFRS) and accommodated by gravity resisting systems. Rotation-permitted foundations could be designed as either capacity protected (CP) or not-capacity protected (NCP).

2.3.3.1 Capacity-protected (CP) foundations

Different types of foundations can be categorized based on their capacity to resist overturning. A capacity-protected foundation is designed to withstand the full capacity of a SFRS and as such does

not develop any plasticity neither in foundation reinforcement nor in the adjacent soil. For such foundation it is possible to determine the associated rotations using approximate methods. This analysis considers how stress is spread in the soil or rock beneath the foundation, as well as how stiff the soil or rock is.

Two important parameters are used in the equation. The first parameter reflects the initial linear rotational stiffness of foundations based on Gazetas (1991) and depends solely on the geometry of the footing. The second factor shows additional turning caused by movements at the "toe" of the foundation, which happen because the soil's stiffness is reduced. It is affected by the magnitude of the bearing stress, which is expressed as a portion of the factored bearing strength, as well as the length-to-width ratio of the distribution of the bearing stress. (Adebar P, 2015).

When the applied overturning moment is sufficiently large to cause the "heel" of a footing to uplift ($M_f \geq P_f l_f / 6$) the rotation of the foundation can be estimated from:

$$\theta = 0.3 \left(\frac{q_s}{G_0} \right) \left(\frac{l_f}{a_s} \right) \left\{ 1 + 2 \left(\frac{a_s}{b_f} \right)^{1.5} \right\} \quad (2.22)$$

where a_s is the length of uniform bearing stress in soil or rock required to resist the factored overturning moment M_f applied to the foundation, given the footing length l_f (perpendicular to axis of rotation). q_s is the magnitude of uniform bearing stress in soil or rock required to balance the applied vertical load P_f , given the footing width b_f (parallel to axis of rotation). G_0 is the initial Shear Modulus of soil or rock.

For buildings of normal importance, like the one in this study, the inter-storey displacement is restricted to 2.5% of the storey height. As outlined in Article 4.1.8.16 of the NBCC 2020, any extra displacements resulting from the foundation's rotation must be included into the seismic load-bearing system's displacements. The total effect of these combined displacements should stay within the predefined limit.

2.3.3.2 Not capacity-protected (NCP) foundations

According to CSA A23.3-19 without rotational restraints can be also designed as no-capacity protected (NCP) foundations, provided that it can be demonstrated that both the SFRS and the elements not designated as part of the SFRS can withstand the increased displacements.

Additionally, foundations held in place by flexible structures can also be designed as NCP foundations, given that the restraining structure can sustain the increased displacements as well.

NCP foundations shall have a factored overturning resistance not less than what is required to resist the factored gravity loads and the larger of

- 75% of the nominal overturning capacity of the SFRS; or
- The overturning moment resulting from design load combinations that include earthquake effects calculated using $R_d R_o$ equal to 2.0.

In the context of a concrete structure like a shear wall, CSA A23.3-19 characterizes the nominal moment as the value achieved when the resistance coefficients (ϕ_c, ϕ_s) are both set to 1.0 and strain hardening ($R_y F_y$) is excluded from consideration. While in the previous editions of S16, the notion of nominal moment was not available, CSA S16-19 introduced a notion of nominal moment, which is calculated using nominal yield stress $R_y F_y$ where R_y is taken equal to 1.

Except as given below, the rotations of NCP foundations designed in accordance with Clause 21.10.3.2.4, CSA A23.3-19 shall be determined using a dynamic analysis that accounts for the reduced rotational stiffness of the footing due to footing uplift and soil deformation.

Instead of using dynamic analysis to determine the increased drifts due to rotations of NCP foundations, the inter-storey drift ratio of the building determined from a fixed-base model can be increased at every level, including immediately above the footing, by a value equal to the largest of

- 50% of the displacement at the top of the SFRS determined from a fixed-base model, divided by the height above the footing.
- the rotation of the foundation calculated using Equation 2.22 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.

The values obtained for drifts using this approximate approach are upper-bound values that can be used to avoid calculating the actual drift ratios in those cases where the SFRS and the members not considered part of the SFRS can easily tolerate the imposed drifts.

2.4 Soil-structure interaction

Soil-structure interaction (SSI) refers to the mutual influence and dynamic behaviour between a structure and the underlying soil or ground during seismic events Figure 2-15. The behaviour of soil significantly influences the response of structures during an earthquake. Different types of soil, such as stiff clay, loose sand, or rock, have varying levels of stiffness, damping, and amplification of ground motions. As explained in the previous sections, in seismic design, dynamic analysis is most often used to model the behaviour of structures under earthquake loads. Traditional methods often assume rigid ground support, neglecting the effects of soil flexibility. However, in SSI analysis, the interaction between the building's dynamic response and the deformable soil is considered. This interaction can lead to changes in the building's natural period, damping ratios, and overall seismic response. Soft soils can amplify ground motions, potentially causing greater vibrations in the structure. Conversely, some soils can damp or absorb seismic energy, reducing the vibrations transferred to the building.

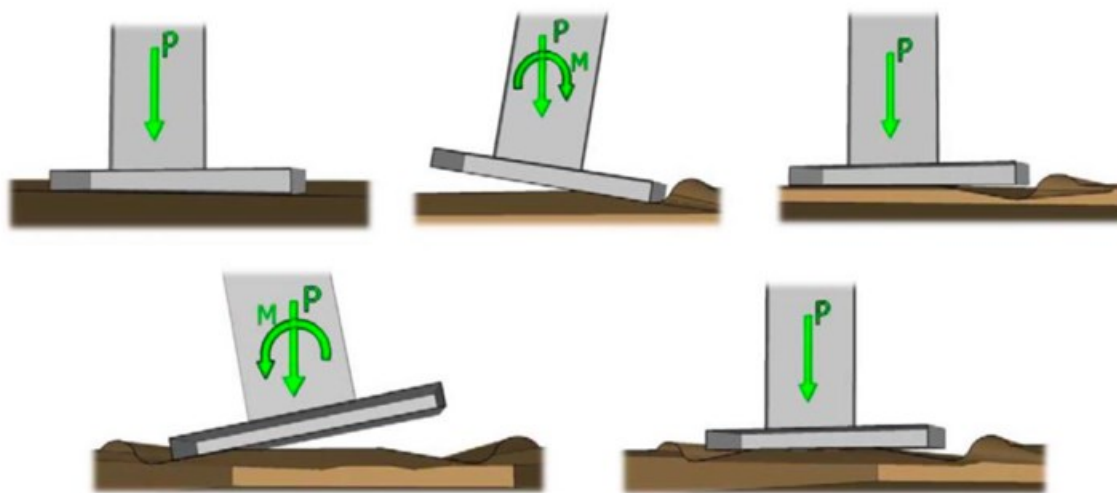


Figure 2-15 Soil-structure interaction (Lemieux 2014)

2.4.1 Definition of soil-structure interaction

In a general context, any analysis where both the structure and soil are concurrently examined, whether under static or dynamic conditions, can be classified as soil-structure interaction analysis. Yet, in a specialized and more precise interpretation, this term is specifically reserved for dynamic analyses where the structure and the soil environment function as an interconnected system during vibrations, and their vibration modes exert mutual influence. In such cases, seismic waves propagate from the soil to the structure, inducing structural vibrations. Simultaneously, the waves reflecting back from the vibrating structure into the ground generate new seismic stresses within the soil. Moreover, the type of foundation, along with its geometric properties and operational depth, will exert an influence on the manner in which the structure's vibrations are experienced.

2.4.2 Interaction mechanisms

SSI consideration can be applied to any building, but its impact can vary significantly. According to NIST 2012 and FEMA 2020, the inclusion of SSI is particularly important for buildings with a large footprint, deep foundations, and a high structure-to-soil stiffness ratio. Figure 2-17 illustrates how SSI can affect a building's response. The high structure-to-soil stiffness ratio can indicate whether the soil will influence the superstructure's deformations.

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:

$$\frac{h'}{v_s T} > 0.1 \quad (2.23)$$

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 analyzed by Bielak (1974), Veletsos and Nair (1975), and Dobry (2014). The term h'/T denotes the structural stiffness in velocity units, facilitating a comparison with soil wave velocities. Essentially, it represents the ratio of structure-to-soil stiffness. Figure 2-16 demonstrates that, for fixed values of h' and v_s , a decrease in T leads to a larger ratio, making the structure more prone to SSI effects. In softer soils, this ratio increases, highlighting a more pronounced impact of SSI on the structural response.

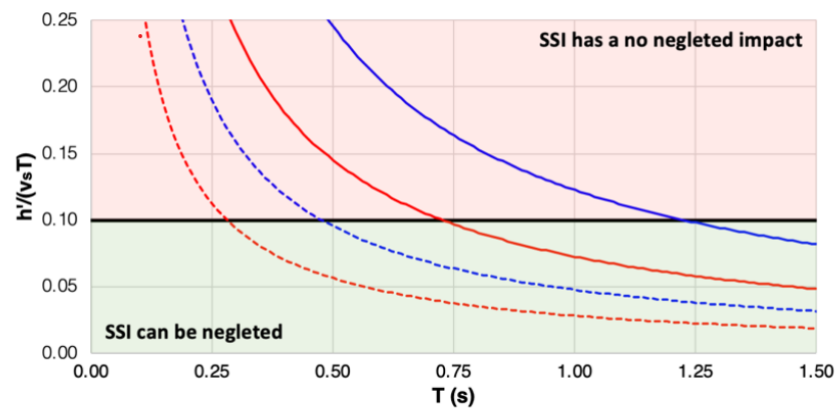


Figure 2-16 Structure-to-soil stiffness ratio for studies scenarios.

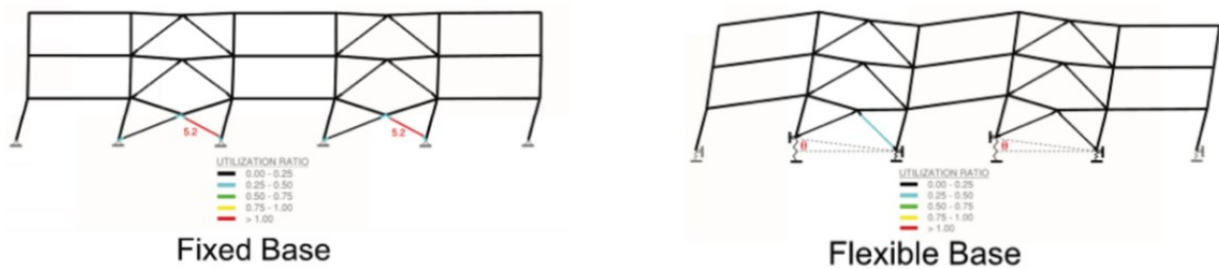


Figure 2-17 The significant impact of soil flexibility on a coupled CBF can change the demand-to-capacity ration of structural elements. Adapted from FEMA (2019).

Stewart et al. (1999) conducted measurements on buildings subjected to ground shaking and found that the structure-to-soil stiffness ratio is the most significant and sensitive parameter for estimating the impact of SSI on seismic response. They concluded that this ratio can be disregarded if it is below 0.1.

There are two common methods for incorporating SSI into analysis: (1) the Direct Analysis Approach (DAA) and (2) the Substructure Approach (SSA). These methods are depicted in Figure 2-18 and Figure 2-19 respectively.

In the DAA, the entire system is modeled using the finite element method. Soil is represented as a continuum medium, employing shell elements for 2D analyses or solid elements for 3D analyses, with linear representations of soil properties. A detailed mesh is used around the foundation, while a coarser mesh is applied to describe the soil response at the model's edges, where boundary elements are located.

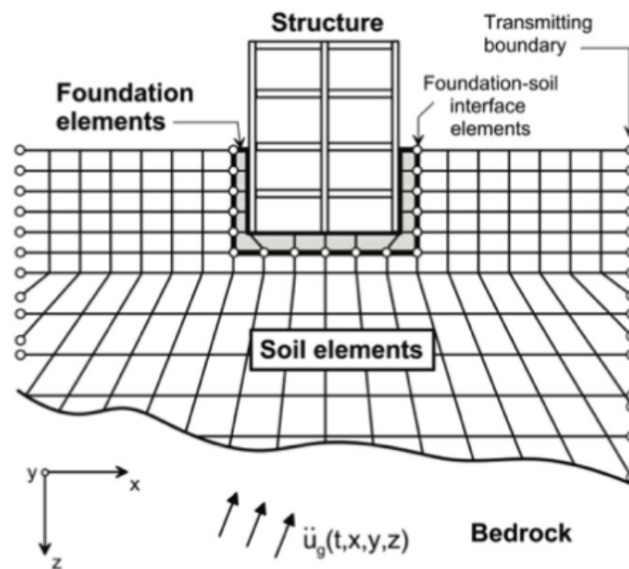


Figure 2-18 Schematic illustration of DAA using continuum modeling by finite elements.
Adapted from NIST 2012.

Due to the computational complexity and challenges in incorporating kinematic interactions caused by spatially variable input motions, this method is infrequently used in practice. Engineers often prefer the Substructure Approach (SSA) instead. More detailed information on the Direct Analysis Approach (DAA), can be found in NIST 2012.

The SSA method is simpler and more practical to implement. In this approach, the soil is represented by a series of springs beneath a discretized elastic beam that symbolizes the foundation. The superstructure is connected to this beam, which in turn is linked to a fully restrained node.

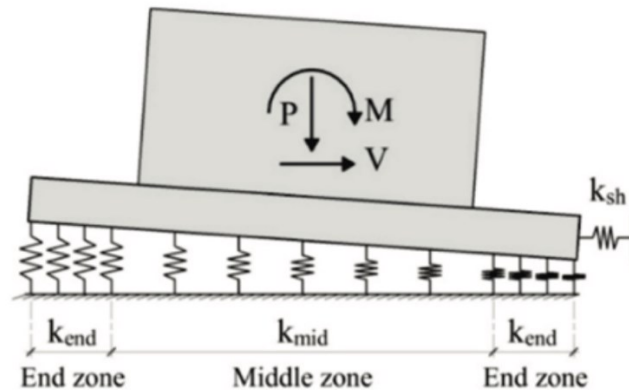


Figure 2-19 Substructure approach using springs that characterize the soil properties. Adapted from FEMA (2020).

2.4.3 Beam On Non-linear Winkler Foundation (BNWF)

One of the most widely used methods for modeling in the Substructure Approach is the Beam On Non-linear Winkler Foundation (BNWF) model. This approach involves modeling the foundation as a discretized system supported by a series of finite stiffness non-linear springs rather than assuming it has fully rigid support. The properties of these springs are adjusted based on soil stiffness and foundation geometry. Constructing the numerical model is relatively straightforward because the springs operate with a single degree of freedom, and the foundation is often modeled as an elastic beam element.

Beyond its simplicity, a key advantage of this method is its ability to capture a non-linear soil response, including yielding under compressive stresses, zero response under tensile stresses, as well as rocking and uplifting behaviour.

The concept of using elastic elements supported by springs was originally proposed by Emir Winkler (1867) to compute stress distribution through a flexible medium. Since then, several other formulations have been introduced, including those by McClelland and Focht (1958), Matlock (1970), Boulanger et al. (1999), Harden et al. (2005), and Raychowdhury and Hutchinson (2008), among others. This study utilized the OpenSees program (McKenna, 1997) for structural analyses. Raychowdhury and Hutchinson (2008) integrated the BNWF concept into OpenSees, which was employed in this research. They proposed incorporating coupled spring stiffness to account for vertical and rotational displacements, providing a more realistic representation of foundation rocking behaviour. Further details on the model parameters used in this study will be discussed in the next chapter.

2.4.4 Kinematic interaction

The input motion of a massless foundation-structure system differs from the free-field ground motion due to kinematic interaction. This interaction occurs when seismic waves from the free field encounter the relatively rigid foundation material, causing the waves to diffract around the foundation due to the high rigidity ratio between the soil and the structure. Additionally, spatial variability in ground motions within the foundation's footprint scatters the energy input into the system, a phenomenon known as base-slab averaging. Ground motions at the foundation's depth also differ from those at the surface because of attenuation through the soil column. Another key aspect of kinematic interaction is the natural filtering of seismic waves with wavelengths comparable to or greater than the footing length, due to the overlapping of waves along the footing (Housner 1957). Kramer and Stewart (2004) demonstrated that this interaction generally reduces the seismic demands on the foundation-structure system. To remain consistent with previous studies (Ushathan 2018), this study does not consider kinematic interaction.

2.4.5 Inertial interaction

Inertial interaction refers to the dynamic interplay between the structure, foundation, and surrounding soil resulting from the motion of the foundation. The foundation's motion causes base shears and overturning moments in the structure as base accelerations are transmitted into the soil. This interaction induces stresses and deformations in the surrounding soil, leading to changes in the overall system's response, such as an increase in period length and the development of

additional damping mechanisms (radiation and soil damping). This phenomenon is depicted in Figure 2-20.

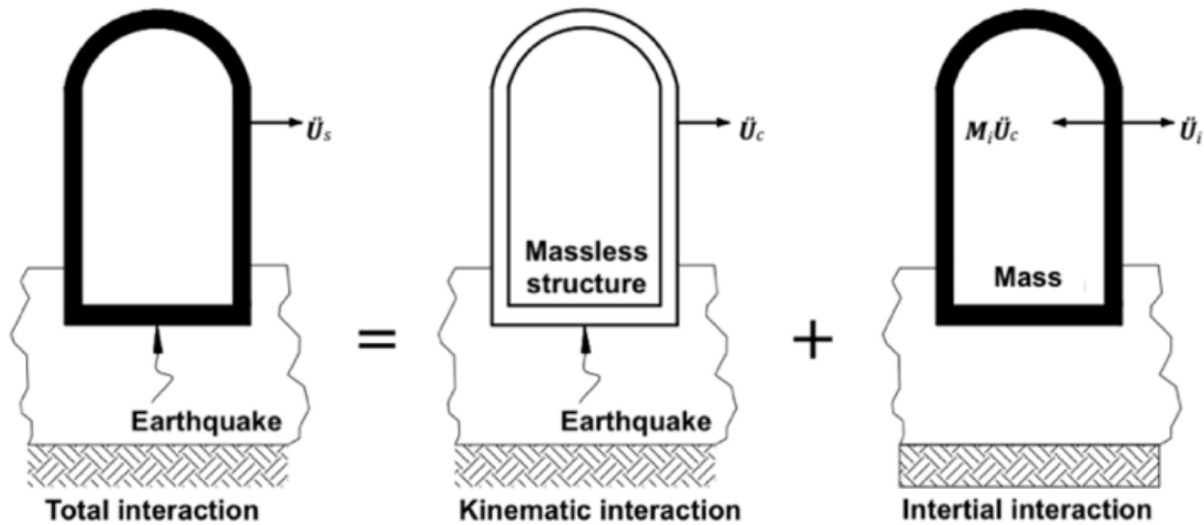


Figure 2-20 Total interaction is given as the sum of kinematic interaction and inertial interaction. Adapted from CFE (2017).

A rigid base is characterized by soil supports that possess infinite stiffness, meaning they lack soil springs. A rigid foundation consists of foundation components with limitless stiffness, meaning they cannot deform. A fixed base, on the other hand, comprises both rigid foundation elements and a rigid base. In contrast, a flexible base analysis takes into account the compliance, or deformability, of both the foundation elements and the soil (NEHRP 2012).

Imagine a single-degree-of-freedom structure with stiffness denoted as ' k ' and mass as ' m ,' which is positioned on a stationary base, as illustrated in Figure 2-21a. The application of a static force, ' F ,' results in a deflection marked as ' Δ .'

$$\Delta = \frac{F}{K} \quad (2.24)$$

In the field of structural dynamics, Clough and Penzien (1993) have provided the formulas for the undamped natural frequency of vibration, represented as ' ω ,' and the period ' T ' of the structure as:

$$\omega = \sqrt{\frac{k}{m}}, \quad T = \frac{2\pi}{\omega} = 2\pi \sqrt{\frac{m}{k}} \quad (2.25)$$

By the combination of equation 2-19 and 2-20 an expression for the square of period is attained as:

$$T^2 = \frac{(2\pi)^2 m}{\frac{F}{\Delta}} = \frac{(2\pi)^2 m \Delta}{F} \quad (2.26)$$

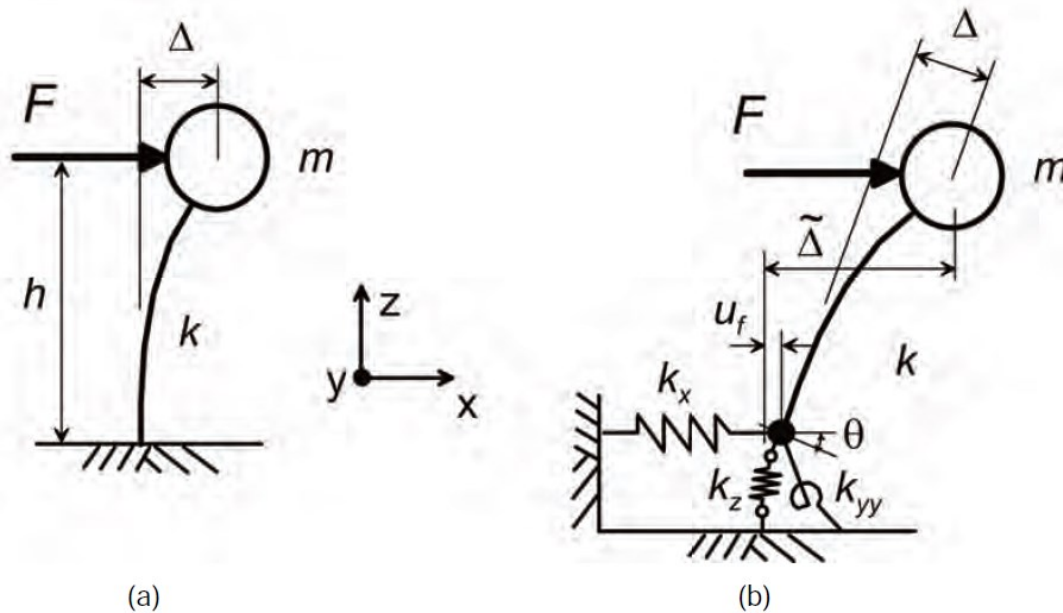


Figure 2-21 : Schematic illustration of deflections caused by force applied to: (a) fixed-base structure; and (b) structure with vertical, horizontal, and rotational flexibility at its base. (NEHRP 2012)

Now, consider the identical structure but with the inclusion of vertical, horizontal, and rotational springs at its base, symbolizing the influence of soil flexibility in contrast to a rigid foundation. This configuration is illustrated in Figure 2-21. The stiffness of the vertical spring in the z direction

is designated as ' k_z ,' the stiffness of the horizontal spring in the x direction is denoted as ' k_x ,' and the rotational spring is labeled as ' k_{yy} ,' indicating rotation in the x-z plane (around the y-y axis). When a force, denoted as ' F ,' is exerted on the mass in the x-direction, the structure undergoes deflection, similar to what occurs in the fixed-base system. However, it's important to note that in this case, the base shear (F) causes a deflection of the horizontal spring by ' u_f ,' and the base moment ($F \times h$) causes a deflection of the rotational spring ' θ .' Consequently, the overall deflection concerning the free field at the top of the structure, represented as ' $\tilde{\Delta}$,' can be expressed as follows:

$$\begin{aligned}\tilde{\Delta} &= \frac{F}{k} + u_f + \theta \cdot h \\ \tilde{\Delta} &= \frac{F}{k} + \frac{F}{k_x} + \left[F \cdot \frac{h}{k_{yy}} \right] h\end{aligned}\quad (2.27)$$

If Equation 2-25 is merged into Equation 2-26, an statement for flexible base period, $\tilde{T}$, is gained as:

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

Combining expressions in Equation 2-24 and Equation 2-27 results in:

$$\left[\frac{\tilde{T}}{T} \right]^2 = \frac{k}{m} \cdot m \cdot \left[\frac{1}{k} + \frac{1}{k_x} + \frac{h^2}{k_{yy}} \right] \quad (2.29)$$

Equation 2-28 transforms into a traditional equation for extending the period, as presented by Veletsos and Meek in their 1974 work.

$$\frac{\tilde{T}}{T} = \sqrt{1 + \frac{k}{k_x} + \frac{k h^2}{k_{yy}}} \quad (2.30)$$

To apply Equation 2-29 to multi-degree-of-freedom structures, you can use the value ' h ' as the height of the center of mass for the first-mode shape. This is commonly known as the effective modal height, which is typically around two-thirds of the total structure height and is defined as 0.7 times the height according to ASCE/SEI 7-10 (ASCE, 2010). In these scenarios, the concept of period lengthening specifically refer to the first-mode period.

2.4.6 The effect of soil-structure interaction

The effect of soil-structure interaction on the parameters of the structure are as follow:

Damping

Damping in a flexible environment is more important than in a rigid environment. Damping in the flexible environment can be divided into two parts, radiation damping and soil hysteresis damping.

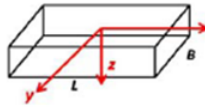
Radiation damping: During an earthquake, after the propagation and vibration of the structure, the waves move from the base of the structure towards the semi-infinite environment of the soil, and the energy is damped due to the collision with this infinite environment. It is worth mentioning that radiation damping increases in softer and deeper soils.

Soil hysteresis damping: This type of damping takes place due to the change of the inelastic shapes of the ground in the vicinity of the foundation, and the greater the intensity of the earthquake, the more these damping increases depending on the type of soil. Considering the interaction of soil and structure usually increases the damping ratio in structures, and increasing the damping ratio in the structure causes the acceleration and displacement of the structure to decrease.

Soil vertical stiffness

The vertical and lateral stiffness can be determined using equations provided by Gazetas (1991a) or as recommended in ATC-40 (1996) Table 2-1.

Table 2-1 Equations for determining shallow foundation stiffness after Gazetas (1991a) as summarized in ATC-40 (1996).

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	$K_{\alpha x}' = \frac{G}{1-\nu} I_x^{0.75} \left(\frac{L}{B} \right)^{0.25} \left(2.4 + 0.5 \frac{B}{L} \right)$
Rotation about y-axis	$K_{\alpha y}' = \frac{G}{1-\nu} I_y^{0.75} \left[3 \left(\frac{L}{B} \right)^{0.15} \right]$
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	$e_{\alpha x} = 1 + 2.52 \frac{H}{B} \left(1 + \frac{2H}{B} \left(\frac{d}{D_f} \right)^{-0.2} \left(\frac{B}{L} \right)^{0.5} \right)$
Embedment Factor, Rotation about y axis	$e_{\alpha y} = 1 + 0.92 \left(\frac{2H}{L} \right)^{0.60} \left(1.5 + \left(\frac{2H}{L} \right)^{1.9} \left(\frac{H}{D_f} \right)^{-0.60} \right)$
Where, K_i = Uncoupled Total Surface Stiffness for a rigid plate on a semi-infinite homogeneous elastic half-space, e_i = Stiffness Embedment Factor for a rigid plate on a semi-infinite homogeneous elastic half-space, L = foundation length, B = foundation width, D_f = depth of embedment, H = foundation thickness, G = Shearing Modulus, ν = Poisson's ratio.	

2.5 Numerical modeling

To conduct a nonlinear time history analysis, it is crucial to define the sections and elements in a way that accurately represents the real behaviour and response of the structure. This involves selecting appropriate material properties, modeling nonlinearities such as plastic hinges or cracking, and ensuring that the interaction between different components, such as beams, columns, braces and connections, is captured accurately. Proper definition of these parameters allows for a more realistic simulation of how the structure will respond under seismic loads, capturing both the elastic and inelastic behaviors, and providing insights into potential failure mechanisms and performance levels.

2.5.1 STKO

STKO, also known as the Scientific ToolKit for OpenSees, serves as a sophisticated Graphic User Interface (GUI) for OpenSees. STKO provides the capability to fully utilize the robust features of OpenSees. It distinguishes itself as the exclusive pre- and post-processing tool that encompasses every available material, element, condition, and interaction within OpenSees, enabling users to tap into the full range of potential that OpenSees offers.

While developing the model through STKO's graphical interface, the software generates both an input file (utilizing Tool Command Language, TCL) and an output file that contains the model's information. Once the model is constructed, one can initiate and execute the analysis. The software will construct an output database based on HDF5, containing the results of the analysis, which can be accessed and manipulated in the post-processing stage. The Python scripting interface provides the flexibility to customize and program the already robust pre and post-processing capabilities as required.

2.5.2 Superstructure modeling

The initial analytical model for the EBFs was created with the ANSR-1 software (Ricles, 1988; Mondkar and Powell, 1975). The software version employed was compiled in 1996 and is not compatible with Windows due to the use of a Fortran compiler. Consequently, to use this software, one must have a DOS operating system, which can present a significant challenge in terms of accessibility.

The other software which was used by Prakash et al, 1993 is the Drain-2DX. The Drain-2DX software has been a key tool in numerous studies related to EBFs, such as those conducted by Richards and Uang (2003) and Rossi and Lombardo (2007). It provides nearly identical analysis capabilities as ANSR-1 but incorporates some extra components, such as an element with zero length. The primary motivations for utilizing this software are its compatibility with the Windows operating system and the opportunity to draw comparisons with the extensive history of prior EBFS studies conducted using this software. (Koboevic et al, 2012)

A third modeling tool for analysis is the OpenSees software, which stands for Open System for Earthquake Engineering Simulation (Mazzoni et al, 2007). Similar to ANSR-1 and Drain-2DX, OpenSees utilizes the finite element method. This software enables the modeling of both linear and non-linear systems under static or dynamic loads. It was primarily developed by the Pacific Earthquake Engineering Research (PEER) and is an open-source code. This open nature allows for the incorporation of new components and fosters continuous advancement in terms of materials, elements, and analysis techniques.

2.5.2.1 OpenSees materials and elements

OpenSees has wide variety of materials and elements, some of them which were used in this research are mentioned as follows:

Elements: Elastic column/beam element, Non-linear column/beam element, Element of zero length. The nonlinear column/beam element available in OpenSees can be defined as:

Force control:

Non-linear Beam Column (the plasticity is distributed along the element Figure 2-22)

Beam With Hinges (plasticity is concentrated on hinges of a length predetermined at the ends with an elastic part between these ball joints)

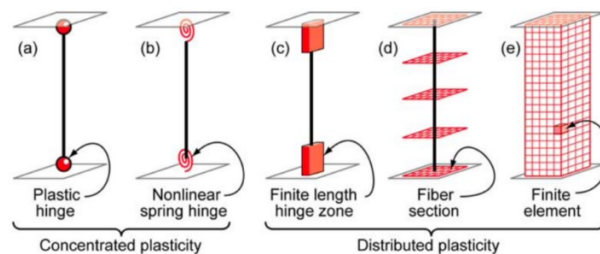


Figure 2-22 Concentrated and distributed plasticity. (NEHRP 2010)

Displacement control:

Disp Beam Column (the plasticity is distributed according to a linear distribution)

In the context of finite element analysis and structural engineering, "force-based elements" (FBE) and "displacement-based elements" (DBE) are two different approaches for modeling and analyzing structures in OpenSees. These approaches determine how the structural elements, such as beams, columns, and trusses, are represented and analyzed within the finite element framework. In FBE approach, the element equations are formulated in terms of forces and their equilibrium equations, often involving axial, shear, and moment forces. FBE is particularly useful when the internal forces are known or can be easily determined and when the displacements are less important for the analysis, such as in materials testing or some specialized analyses. Force-based elements are suitable for problems where stress distributions and stress concentrations are of primary interest.

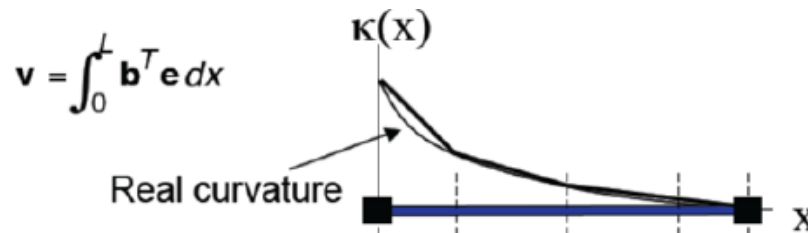


Figure 2-23 FBE solution compared with the exact solution. (Terzic 2011)

In DBE approach, the element equations are formulated in terms of nodal displacements, and the focus is on solving for the displacement field across the element. DBE is commonly used in most structural engineering analyses, including linear and nonlinear static and dynamic analyses. This approach is well-suited for problems where the deformation and displacements of the structure are the primary focus.

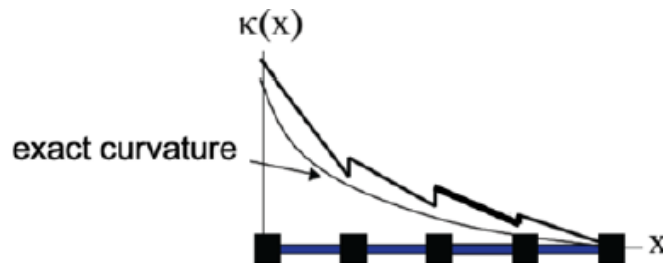


Figure 2-24 DBE solution compared with the exact solution. (Terzic 2011)

The main advantage of the force control element compared to displacement control is the possibility of using a single element to represent the behaviour of a column/beam member.

Material: Elastic material (elastic, elastic-plastic or elastic without tension), Material with hardening state (kinematic and isotropic hardening), Concrete material (zero resistance in traction (Concrete01), resistance in linear traction (Concrete02) or non-tensile resistance linear (Concrete03)), Steel material (bilinear resistance with hardening (Steel01) or model of Giuffre-Menegotto-Pinto (Steel02)), Hysteretic material, Viscous material. It is also possible to combine several materials in order to make them act in parallel or in series.

The text discusses a unique feature of OpenSees, where users have the flexibility to independently choose the discretization of elements in sections and the associated material behaviour. It's possible to refine modeling at various levels, including element type, section, and material. The characteristics of non-linear Beam Column elements are influenced by integration points, section type, material, coordinate transformation, maximum iteration count, and tolerance. Integration along the element follows Gauss-Lobatto quadrature, considering two points at the ends (Figure 2-25). For the central segment of the beam With Hinges element, it behaves as elastic linear and shares properties with the elastic column/beam element. At the ends, it resembles the non-linear Beam Column element, with an added specification for the length of the plastic hinge at each end.

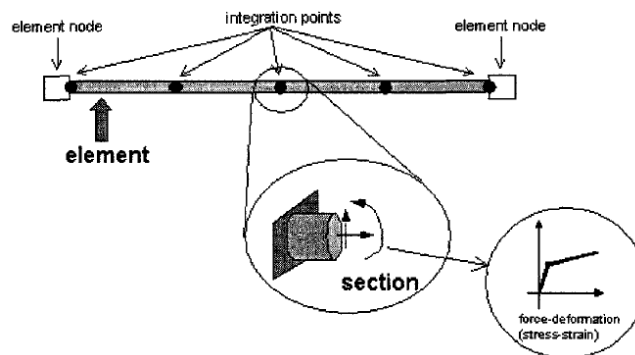


Figure 2-25 Modeling of a nonlinear element (Mazzoni et al. 2007)

CHAPTER 3 PROTOTYPE BUILDING DESIGN

To examine the influence of soil-structure interaction on foundation sizing and the structural behaviour 3- and 8-storey chevron-type steel eccentrically braced frames (EBFs) were designed for Vancouver and Montreal on site categories C and E, following the requirements NBCC 2020 and CSA S16-19 provision. Thus, in total 8 different braced frames were designed. The layout was adopted from Muruganathan (2018) and Reyes Fernandez (2023) who carried out the study on seismic design of foundation of concentrically steel braced frames in Canada.

3.1 Building description

The buildings illustrated in Figure 3-1 are traditional office building 32.5 m wide and 54.5 m long. The longer side of the buildings is positioned in East-West direction. The storey height for the first level is 4.5 m and for the upper floor is 4.0 m giving the total height of 12.5 m and 32.5 m for 3- and 8-storey buildings respectively. It is assumed that the slab is rigid and can transfer the lateral loads to EBFs.

The typical layout for the buildings in Montreal and Vancouver are illustrated in Figure 3-2 a and Figure 3-2 b respectively. The EBFs which are subject of this research are placed in the N-S direction. Two centrally located EBFs are positioned in Montreal's building in a symmetrical manner. Due to the significantly higher level of seismic activity in Western Canada compared to Eastern Canada, two additional EBFs are added in the building's perimeter in the N-S direction for Vancouver. Lateral loads in the E-W direction are carried by the moment-resisting frames situated along the perimeter.

All vertical loads are consistent between Montreal and Vancouver, except for the snow and cladding loads. In Montreal, the snow load is 2.48 kPa, while in Vancouver, it's 1.64 kPa. Similarly, for cladding loads, for Montreal 1.50 kPa is considered (brick), whereas for Vancouver 1.00 kPa is applied (glass curtain).

All structural elements should be designed in accordance with the load combinations defined by NBCC 2020, as follows:

Dead load	1.4 D
Live load	1.25 D + 1.5 L + 1.0 S

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

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

Lateral earthquake $D + E$

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

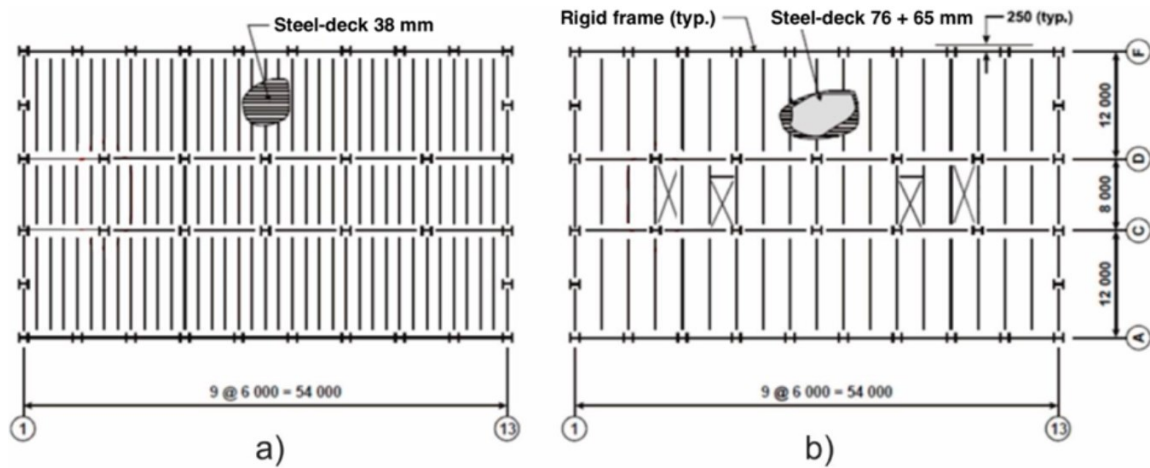


Figure 3-1 Building lay-out (Tremblay 2017 and Murugananthan 2018)

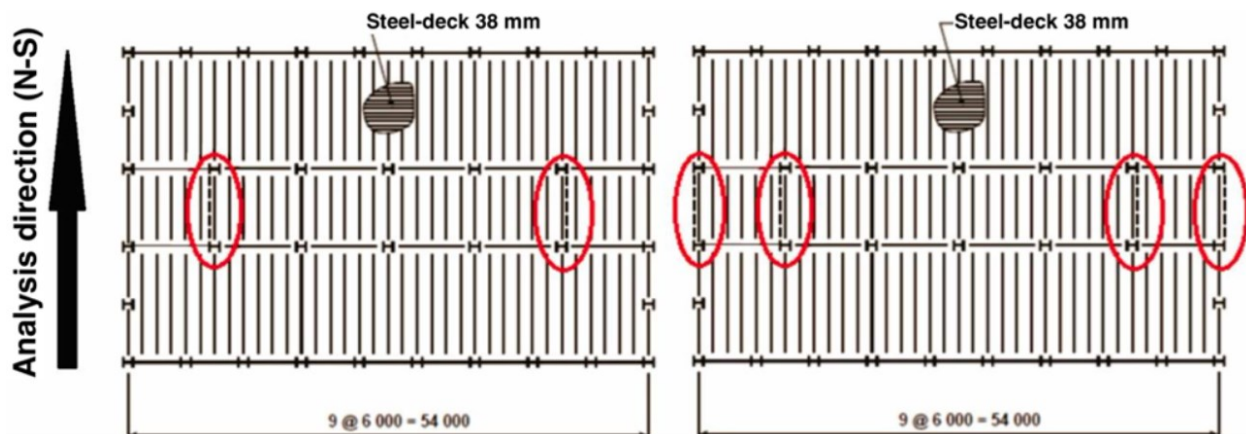


Figure 3-2 : Location of EBFs for Montreal Scenarios (a) and for Vancouver scenarios (b).

(Murugananthan 2018)

3.2 Design assumptions

- The study was done in north-south direction.

- For Montreal two braced frames will take seismic load and accidental torsion while for Vancouver due to the larger magnitude of seismic loads, four braced frames will take seismic load.
- According to CSA S16-19, 0.005 gravity load was considered in addition to the seismic loads.
- To take into account P- Δ effects, the U2 amplification factor was calculated, and the seismic loads were amplified where $U2 > 0.10$.
- Linear Dynamic Analysis was done on 2D model in SAP2000 according to Response Spectrum method and NBCC 2020.
- SFRS Ductility-Related Force Modification Factors, $R_d = 4$, Overstrength-Related Force Modification Factors, $R_o = 1.5$.
- The load pattern is as follows:

Table 3-1 Dead and Live loads for Montreal and Vancouver

Load pattern	Unit	Montreal	Vancouver	Comments
Dead roof	kPa	1.35	1.35	
Dead floor	kPa	3.60	3.60	
Partition	kPa	1.00	1.00	Taken 0.5 kPa to calculate seismic mass
Live load	kPa	2.40	2.40	
Snow load	kPa	2.48	1.64	Taken 25 % to calculate seismic mass
Exterior walls	kPa	1.50	1.00	

3.3 Seismic hazard values

Seismic hazard values were obtained from the Seismic hazard calculator: <https://earthquakescanada.nrcan.gc.ca/hazardalea/interpolated/nbc2020-cnb2020-en.php>.

Table 3-2 The seismic hazard values NBCC 2020 for Vancouver and Montreal

Seismic Zone	Soil Type	Sa(0.2) [g]	Sa(0.5) [g]	Sa(1.0) [g]	Sa(2.0) [g]	Sa(5.0) [g]	Sa(10.0) [g]	PGA [g]	PGV [m/s]
Montreal	C	0.839	0.489	0.257	0.116	0.03	0.00969	0.46	0.337
Probability (% in 50 years) = 2%	E	0.76	0.674	0.407	0.198	0.0556	0.0169	0.46	0.48
Vancouver	C	1.1	0.881	0.511	0.311	0.0881	0.0372	0.477	0.532
Probability (% in 50 years) = 2%	E	1.12	1.23	0.951	0.605	0.188	0.067	0.522	0.858

The seismic hazard values in NBCC 2020 were increased noticeably in compare with NBCC2015 which may lead to higher demand in section designs and shorter modal period T_{modal} .

Table 3-3 Comparing between the seismic hazard values of NBCC 2020 and NBCC 2015

Seismic Zone	Soil Type	Sa(0.2) [g]	Sa(0.5) [g]	Sa(1.0) [g]	Sa(2.0) [g]	Sa(5.0) [g]	Sa(10.0) [g]	PGA [g]	PGV [m/s]
Montreal 2020	C	0.839	0.489	0.257	0.116	0.03	0.00969	0.46	0.337
Montreal 2015	C	0.595	0.31	0.148	0.068	0.018	0.006	0.377	0.255
Ratio	C	1.41	1.58	1.74	1.71	1.67	1.62	1.22	1.32
Vancouver 2020	C	1.1	0.881	0.511	0.311	0.0881	0.0372	0.477	0.532
Vancouver 2015	C	0.847	0.754	0.425	0.257	0.081	0.029	0.368	0.552
Ratio	C	1.30	1.17	1.20	1.21	1.09	1.28	1.30	0.96

3.4 Calculation of T_{emp} , $S(T_a)$, V_{ESF} , V_{DYN}

Seismic base shears were determined using response spectrum analysis. Base shear was first determined using static equivalent method, T_{emp} , $S(T_a)$, and V_{ESF} were calculated considering the soil type and seismic zone. V_{ESF} underwent amplification to factor in accidental torsion. The structural designs were initially performed using spreadsheets, followed by the creation of structural models in SAP2000 to determine modal periods and adjust base shear value as permitted by NBCC2020.

The response spectrum results were calibrated based on V_{DYN} and V_{ESF} to validate the real inter-storey drifts. The lowest and highest values for the V/W ratio are respectively found in the 8-storey braced frame in Montreal on soil type C (0.028) and the 3-storey frame in Vancouver on soil type E (0.209).

Table 3-4 The ratio between seismic shear force and the total weight of the building

Zone	Levels	Site Class	2 * Temp (s)	T1 _{modal} (s)	S(T_a)	W (kN)	V _{ESF} (kN)	V/W
Montreal $R_d R_o = 6$	3	C	0.625	0.689	0.431	20688	1486	0.072
		E	0.625	0.61	0.587	20688	2024	0.098
	8	C	1.625	1.81	0.169	62218	1751	0.028
		E	1.625	1.626	0.276	62218	2866	0.046
Vancouver $R_d R_o = 6$	3	C	0.625	0.54	0.850	19425	2751	0.142
		E	0.625	0.45	1.256	19425	4067	0.209
	8	C	1.625	1.33	0.445	59587	4419	0.074
		E	1.625	0.98	0.951	59587	9444	0.158

It is important to calibrate the target spectrum considering accidental torsion therefore amplification factor is multiplied to V_{ESF} before calibration. V_{ESF} per SFRS are equal to 65% and 32.5% V_{Total} for Montreal and Vancouver respectively Figure 3-3. The structure is regular and V_{ESF} should be multiplied by 0.8 before calibration.

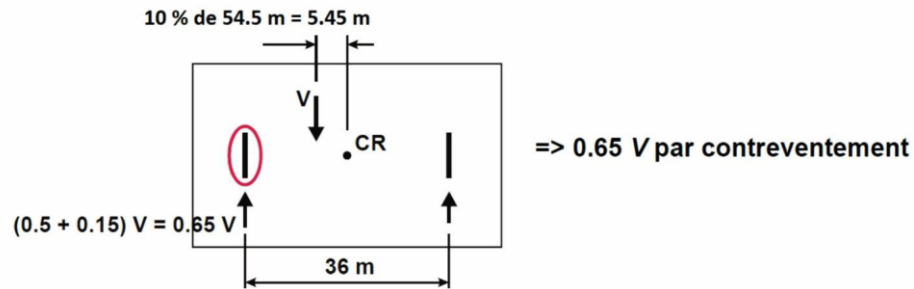


Figure 3-3 Distribution of forces between the bracings including the effect of accidental torsion. (Tremblay 2022)

Table 3-5 Calibration factor

Zone	Levels	Site Class	V_{DYN} (kN)	$V_{DYN}/R_d R_o$	V_{ESF} (kN)	Calibration Factor
Montreal $R_d R_o = 6$	3	C	3741	623.5	719	0.192
		E	5640	940	1052	0.187
	8	C	4206	701	770	0.183
		E	9396	1566	1489	0.167
Vancouver $R_d R_o = 6$	3	C	3948	658	761	0.193
		E	5623	937	1057	0.188
	8	C	5722	954	1148	0.201
		E	11417	1903	2455	0.215

3.5 Frame design

This section details the sequence and primary actions undertaken during the structural design process. All constructions consist of CSA-G40.21-350W steel components, possessing with a yield strength equal to 350 MPa and a modulus of elasticity equal 200,000 MPa.

3.5.1 Ductile link

The initial phase of EBF's design involves selecting a beam for the ductile link. The shear force and moment in the link resulting from seismic loads, denoted as V_{link} and M_{link} , is determined by:

$$V_{link} = \frac{V_x h_x}{L} \quad (3.1)$$

$$M_{link} = \frac{V_{link} e}{2} \quad (3.2)$$

where V_x is the cumulative lateral force at the floor and L is the width of the bracing and e is the length of ductile link.

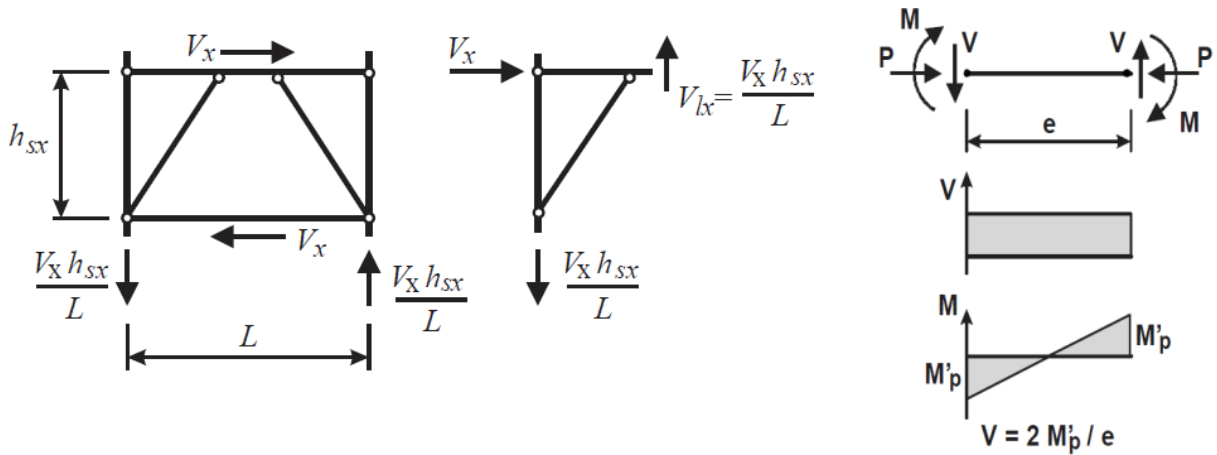


Figure 3-4 Shear force and moment in the ductile link (Tremblay 2022)

Appropriate sections are chosen based on their V_r for shear or bending resistance. 600 mm length was chosen for links making them shear-critical ($e < 1.6M_p/V_p$). It's crucial to select beams ensuring a balanced force-to-link resistance ratio across the building's height, encouraging uniform inelastic demand in links over the height of the frame. The beam's section outside the link matches that of the link

$$\frac{V_{link}}{V_r} = \lambda \leq 1 \quad (2.3)$$

Where:

$$V_r \geq \min \left(\phi V'_p, 2\phi \frac{M'_p}{e} \right) \quad (2.4)$$

$$V'_p = V_p \left[1 - \left(\frac{P_f}{AF_y} \right)^2 \right]^{0.5} \quad (2.5)$$

$$V_p = 0.55A_w F_y \quad (A_w = dw) \quad (2.6)$$

$$M'_p = 1.18M_p \left(1 - \frac{P_f}{AF_y} \right) \leq M_p \quad (2.7)$$

$$M_p = Z_x F_y \quad (2.8)$$

3.5.2 Other frame members

The sizing of the diagonals and the beam section outside the link should consider forces equivalent to 1.30 times the nominal capacity of the link to account for amplified forces resulting from the link strain hardening.

For outer beams, the critical scenario typically pertains to the interaction between tension and moment (expressed as $T/T_r + M/M_r < 1.0$). As previously mentioned in chapter 2, a moment-resisting connection between the diagonal and the beam was assumed to minimize the moment applied to the beam outside of the link. The link end moment was distributed between the brace and the beam in accordance with equations 2.15 to 2.16.

3.6 Design summary

The summary of the sections selected for all research scenarios is presented in Table 3-6 and Table 3-7.

Table 3-6 Section design for Montreal

Seismic zone : Montreal				Seismic Shear force (V) is divided between 2 EBF			
Levels	Sections			Properties of the links			Δ_x
	Brace	Column	Beam	1.6Mp/Vp e=600mm	V _f /V _r	Y _p (Rad)	(%h _s)
Structure Montreal 3 Soil C (V= 1486 kN - T1 modal=0.689 s - 2T _{emp} =0.625 s)							
3	HSS 203x203x7.9	W310X107	W310X28.3	637	0.39	0.034	0.50
2	HSS 203x203x9.5	W310X107	W310X67	1201	1.00	0.050	0.75
1	HSS 203x203x13	W310X107	W360X101	1513	0.93	0.041	0.61
Structure Montreal 3 Soil E (V= 2024 kN - T1 modal=0.61 s - 2T _{emp} =0.625 s)							
3	HSS 229x229x6.4	W310X117	W310X52	1007	0.52	0.043	0.64
2	HSS 229x229x7.9	W310X117	W360X79	1254	0.93	0.056	0.84
1	HSS 229x229x13	W310X117	W410X114	1472	0.95	0.049	0.83
Structure Montreal 8 Soil C (V= 1751 kN - T1 modal=1.83 s - 2T _{emp} =1.625 s)							
8	HSS 178x178x7.9	W360X64	W310X28.3	637	0.42	0.037	0.56
7	HSS 203x203x7.9	W360X64	W310X28.3	637	0.83	0.047	0.71
6	HSS 229x229x7.9	W360X179	W410X46.1	912	0.86	0.047	0.71
5	HSS 229x229x9.5	W360X179	W460X52	925	0.89	0.041	0.62
4	HSS 229x229x13	W360X179	W460X68	1031	0.85	0.039	0.59
3	HSS 229x229x13	W360X382	W460X82	1174	0.88	0.038	0.57
2	HSS 229x229x13	W360X382	W460X89	1211	0.91	0.037	0.56
1	HSS 229x229x16	W360X382	W460X106	1174	0.87	0.034	0.51
Structure Montreal 8 Soil E (V= 2866 kN - T1 modal=1.58 s - 2T _{emp} =1.625 s) e=600 _{mm}							
8	HSS 178x178x7.9	W360X79	W250X38.5	863	0.75	0.056	1.13
7	HSS 203x203x13	W360X79	W410X46.1	912	0.94	0.064	1.28
6	HSS 203x203x16	W360X216	W410X75	1100	0.96	0.064	1.28
5	HSS 229x229x13	W360X216	W460X89	1211	0.99	0.061	1.23
4	HSS 229x229x16	W360X216	W460X106	1174	0.95	0.054	1.08
3	HSS 229x229x16	W360X463	W460X144	1568	0.98	0.051	1.03
2	HSS 254x254x16	W360X463	W530X123	1310	0.95	0.048	0.95
1	HSS 254x254x16	W360X463	W530X182	1747	0.96	0.042	0.95

Table 3-7 Section design for Vancouver

Seismic zone : Vancouver				Seismic Shear force (V) is divided between 4 EBF			
Levels	Sections			Properties of the links			Δ_x (%h _s)
	Brace	Column	Beam	1.6Mp/Vp e=600mm	V _f /V _r	Yp (Rad)	
Structure Vancouver 3 Soil C (V= 2751 kN - T1 modal=0.519 s - 2T _{emp} =0.625 s)							
3	HSS 203x203x7.9	W360X79	W310X28.3	637	0.43	0.042	0.62
2	HSS 203x203x7.9		W310X52	1007	0.85	0.062	0.93
1	HSS 203x203x13		W360X79	1254	0.93	0.047	0.71
Structure Vancouver 3 Soil E (V= 4067 kN - T1 modal=0.456 s - 2T _{emp} =0.625 s)							
3	HSS 229x229x6.4	W310X117	W310X52	1007	0.54	0.043	0.65
2	HSS 229x229x7.9	W310X117	W360X91	1444	0.94	0.064	0.96
1	HSS 229x229x13	W310X117	W410X114	1472	0.97	0.052	0.87
Structure Vancouver 8 Soil C (V= 4419 kN - T1 modal=1.30 s - 2T _{emp} =1.625 s)							
8	HSS 178x178x7.9	W360X64	W310X28.3	637	0.47	0.061	0.92
7	HSS 203x203x7.9	W360X64	W310X32.7	678	0.86	0.080	1.22
6	HSS 229x229x7.9	W360X179	W460X52	925	0.74	0.080	1.20
5	HSS 229x229x9.5	W360X179	W460X52	925	0.92	0.079	1.19
4	HSS 229x229x13	W360X179	W460X68	1031	0.88	0.067	1.01
3	HSS 229x229x13	W360X382	W460X82	1174	0.90	0.065	0.98
2	HSS 229x229x13	W360X382	W460X89	1211	0.91	0.061	0.92
1	HSS 229x229x16	W360X382	W460X106	1174	0.87	0.055	0.83
Structure Vancouver 8 Soil E (V= 9444 kN - T1 modal=1.00 s - 2T _{emp} =1.625 s) e=600 _{mm}							
8	HSS 178x178x13	W610X101	W410X46.1	912	0.58	0.064	1.28
7	HSS 203x203x13	W610X101	W460X68	1031	0.84	0.077	1.55
6	HSS 229x229x13	W760X284	W530X101	1305	0.88	0.080	1.62
5	HSS 254x254x16	W760X284	W530X123	1310	0.91	0.080	1.61
4	HSS 254x254x16	W760X284	W530X138	1305	0.94	0.076	1.52
3	HSS 305x305x13	W760X582	W530X196	1738	0.92	0.065	1.29
2	HSS 305x305x16	W760X582	W530X219	1731	0.88	0.058	1.16
1	HSS 305x305x16	W760X582	W530X248	1885	0.96	0.053	1.07

3.7 Foundation design

As a result of high ductility $R_d=4$ and $R_o=1.5$ in all cases M_{probable} is less than overturning moment $M_{R_dR_o=2}$ (Table 3-8). Therefore, the foundation design was done according to the capacity protected method. On the other hand, non-capacity protected method can not be used because it leads to bigger dimension for foundation in compare with CP method. Instead of using the non-capacity-protected method, the foundation was designed without any capacity factor, considering loads computed with $R_dR_o=6$, in order to assess whether the inter-storey drift and soil capacity fall within the accepted range of NBCC 2020. When considering the combination of probable moment and gravity loads, two limits must be considered for foundation designs. First, the maximum stress (q_{max}) under the foundation, determined by the triangular stress distribution, should not exceed the ultimate soil capacity. Second, the superstructure displacement, resulting from foundation rotation and inter-storey drift, should be limited to less than 2.5% of the building's height (2.5%h).

Table 3-8 Comparison between M_{probable} and M corresponding rocking limit ($R_dR_o=2$)

Zone	Levels	Site Class	P_f	M_{probable}	$M_{R_dR_o=2}$	$M_{R_dR_o}/M_{pr}$
Montreal	3	C	1899	18573	24054	1.30
		E	1899	23347	34264	1.47
	8	C	6039	64944	82236	1.27
		E	6039	90823	129580	1.43
Vancouver	3	C	1899	16719	23016	1.38
		E	1899	23448	33567	1.43
	8	C	6039	66735	90323	1.35
		E	6039	128608	202757	1.58

3.7.1 Soil properties

To maintain consistency with prior research, these soil properties were obtained directly from Murugananthan (2018). A resistance factor (ϕ) equal to 0.5 is employed to reduce the soil capacity

(q_{ult}) to the design level, q_{fact} , in accordance with the guidelines outlined in the Canadian Foundation Engineering Manual. Table 3-9 provides a list of the soil properties employed in the design.

Table 3-9 Design Soil properties

Site Class	q_{ult} (kPa)	q_{fact} (kPa)	G (MPa)	γ_0 (kN/m ³)
C	3000	1500	100	20
E	400	200	20	20

3.7.2 Foundation demand

The overturning moment demand on the foundation can be determined based on the chosen design method. In the case of a capacity-protected foundation, the overturning moment equals M probable, which results from the yielding of all shear links in this project. On the other hand, the non-capacity factored method involves designing the foundation using a lateral load equal to $V_{ESF} = \frac{S(Ta)MvIEW}{R_d R_o}$ when $R_d R_o = 6$.

As an example, various overturning moment demand for 3-storey building in Montreal site category C could be found in Table 3-10.

Table 3-10 Overturning moment demand

Overturning moment	M (kN.m)
Corresponding to $R_d R_o = 6$	8018
Corresponding to $R_d R_o = 2$	24054
Corresponding to $R_d R_o = 1$	48108
Probable	19322

3.7.3 Summary of footing size

All the foundations were designed to withstand two overturning moment demands, namely the capacity-protected M_{prob} and the one without any capacity factor, $M_{RdRo=6}$, on site Class C and E. The influence of foundation rotation on inter-storey drift was taken into account. The rotation was estimated by this formula $\theta = 0.3 \left(\frac{q_s}{G_0} \right) \left(\frac{l_f}{a_s} \right) \left\{ 1 + 2 \left(\frac{a_s}{b_f} \right)^{1.5} \right\}$ which was mentioned in section 2.2.1. All footing dimensions were summarized for Vancouver in Table 3-11.

Table 3-11 Footing dimensions for Vancouver 8-and 3-storey site class C and E

Storey	Site Class	M_{design}	FOOTING DIMENSIONS			
			h(m)	b(m)	t(m)	Volume(m ³)
3	C	M_{prob}	11.2	4.5	1.3	65.52
		$M_{RdRo=6}$	11.2	2	1.3	29.12
	E	M_{prob}	16.8	6	1.7	171.36
		$M_{RdRo=6}$	16.8	5	1.7	142.8
8	C	M_{prob}	16	6	1.5	144
		$M_{RdRo=6}$	16	3.5	1.5	84
	E	M_{prob}	24	12	2	576
		$M_{RdRo=6}$	24	9	2	432

CHAPTER 4 NUMERICAL MODELING

In this study, 24 two-dimensional models were developed using STKO. As an example, the types of elements and connections are illustrated in the Figure 4-1. The shear links, beams outside the links, and braces were modeled using non-linear elements to capture the plastic deformation in this component. The connection between beams and braces is defined by a zero-length element with high rigidity, which enable moment transfer to the braces and reduces the demand on the beams outside the shear links.

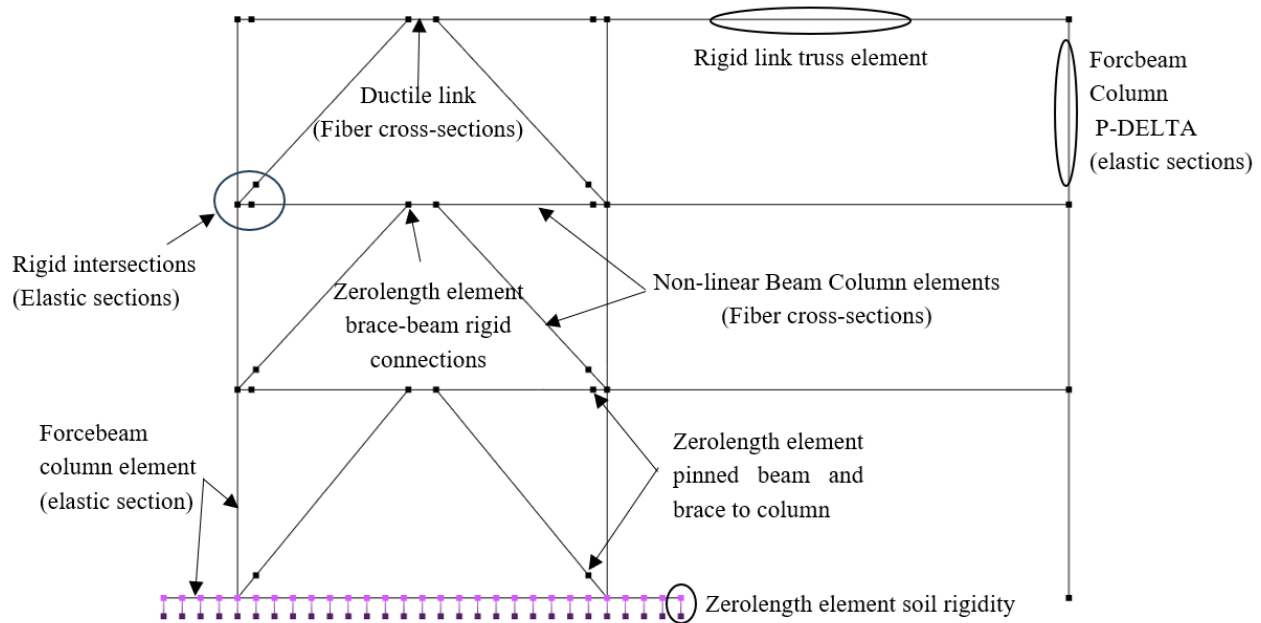


Figure 4-1 Numerical model OpenSees

Members' cross sections were selected using the available Canadian Institute of Steel Construction (CISC) standard shapes and the material properties were modelled using the Steel02 material model in STKO. The yield force was set equal to $V_{pr} = 0.55A_vR_yF_y$ where $F_y=345$ MPa and $R_y= 1.1$ for shear links and beams outside of the link.

The elastic column/beam element is employed for columns that maintain their elasticity throughout the loading process. Its properties in three dimensions are determined by the material moduli (Young's modulus and shear modulus), the cross-sectional area, the moments of inertia

along the two local axes, and in torsion. In this study, this element was used to represent the columns and rigid extensions of the structural members.

The zero-length element serves to replicate the imposed deformation relationship between degrees of freedom (DOFs) of two nodes situated at the same coordinates. Within this project, the zero-length element was employed to model the hinged connections at the intersections of beams and columns, as well as at the bottom of diagonal members and the soil springs.

4.1 Modeling of the ductile link

Numerous analytical models have been created in the past to simulate the nonlinear characteristics of ductile links. One of the most useful models is the one developed by Richards and Uang in 2003, which is rooted in the earlier model by Ramadan and Ghobarah from 1995. The link element consists of an elastic beam fixed at both ends by three bilinear springs of negligible length. These springs operate in parallel to account for shear hardening. When these springs are properly calibrated, this model effectively represents the shear resistance of the link as it undergoes plastic deformation.

As it is shown in Figure 4-2, To represent the multi-linear pattern formed by three shear yielding plates, it is essential to use three bilinear springs placed in parallel at both ends of the link, each with negligible length. This adjusted modeling focuses solely on simulating shear hardening and is most effective for shorter links. Figure 4-3 outlines the specific stiffness values to assign to these three bilinear springs to achieve the desired overall behaviour, as recommended by Richards and Uang in 2003.

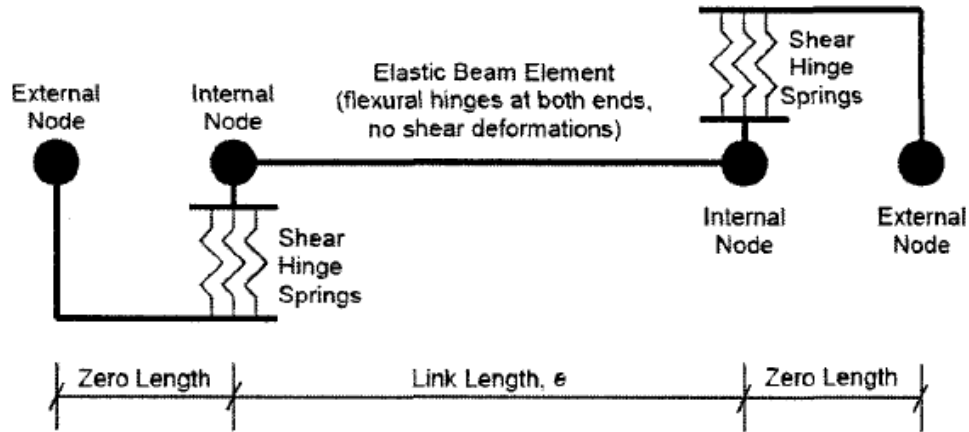


Figure 4-2 Ductile link proposed by Richards and Uang (2003)

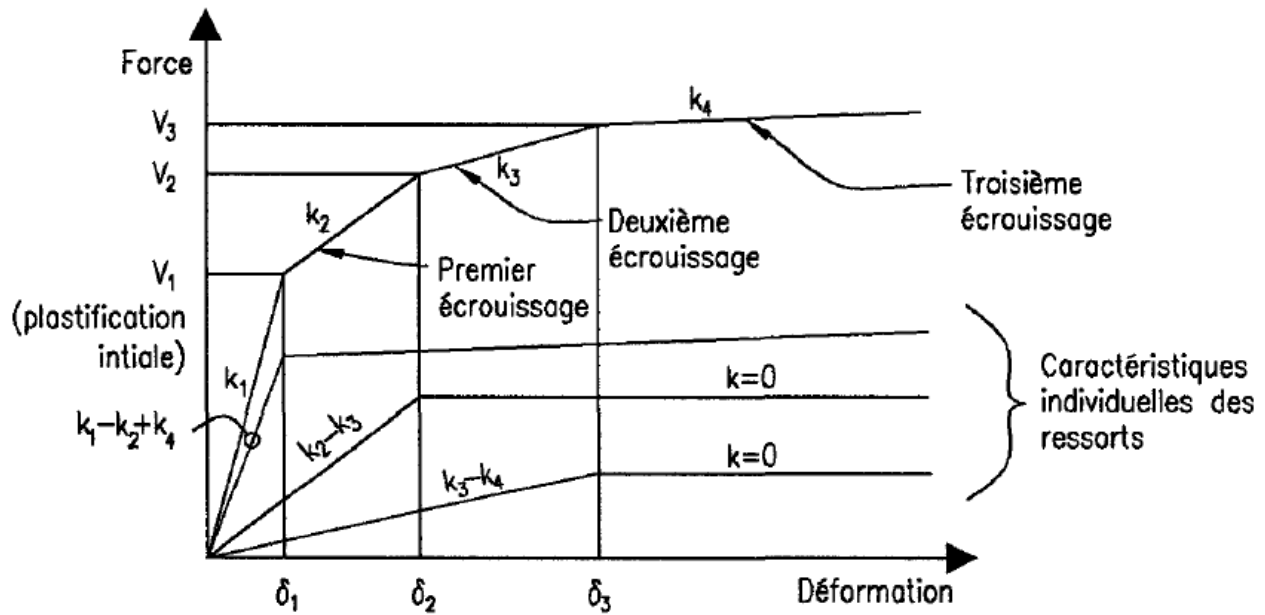


Figure 4-3 Simulation of ductile segment behaviour. (Ramadan and Ghobarah, 1995)

$$K_{v1} = \frac{2GA_{shear}}{e} \quad (4.1)$$

$$K_{v2} = 0.030K_{v1} \quad (4.2)$$

$$K_{v3} = 0.015K_{v1} \quad (4.3)$$

$$K_{v4} = 0.002K_{v1} \quad (4.4)$$

(Koboevic et al, 2012) found three estimates yielding bound for shear resistance which start with probable shear as follow:

$$V_{pr} = 0.55A_v R_y F_y \quad (R_y = 1.1) \quad (4.5)$$

$$1.0V_{pr}, 1.20V_{pr}, 1.35V_{pr} \quad (4.6)$$

To model the shear links behaviour, corresponding stiffness and yielding bond was assigned to each of them.

To achieve a smoother transition into shear plasticity, an alternative modeling approach for the link was created. In this revised OpenSees model, the key difference is the utilization of the Steel02 material in combination with a single spring, as opposed to the three bilinear springs used in the previous model.

In this method force-based fiber elements were used along with steel 2 material, which were further equipped with an additional section aggregator to effectively capture the nonlinear shear characteristics of the links. The shear aggregator itself was constructed in OpenSees using the Steel02 material, with the yield strength input value set to match the link's yield shear force (V_{pr}) and the modulus of elasticity input value reflecting the initial shear stiffness (GA_v), where G represents the shear modulus, and A_v corresponds to the shear area of the link. The strain hardening ratio (b) was established at 0.003. Additionally, the isotropic hardening parameters for the aggregator section, denoted as $a1$, $a2$, $a3$, and $a4$, were set at (0.033, 1, 0.033, and 1.0) respectively. For the elastic-plastic transition, the parameters ($R0$, $cR1$, and $cR2$) were configured as (18.5, 0.925, and 0.15), respectively (T.Y. Yang et al, 2020).

To validate the model, experimental data from tests conducted by Dusicka and Lewis (2010) were employed. Figure 4-4 displays the comparison between the numerical model and the experimental data, showing that the proposed model accurately simulates the nonlinear behaviour of the link.

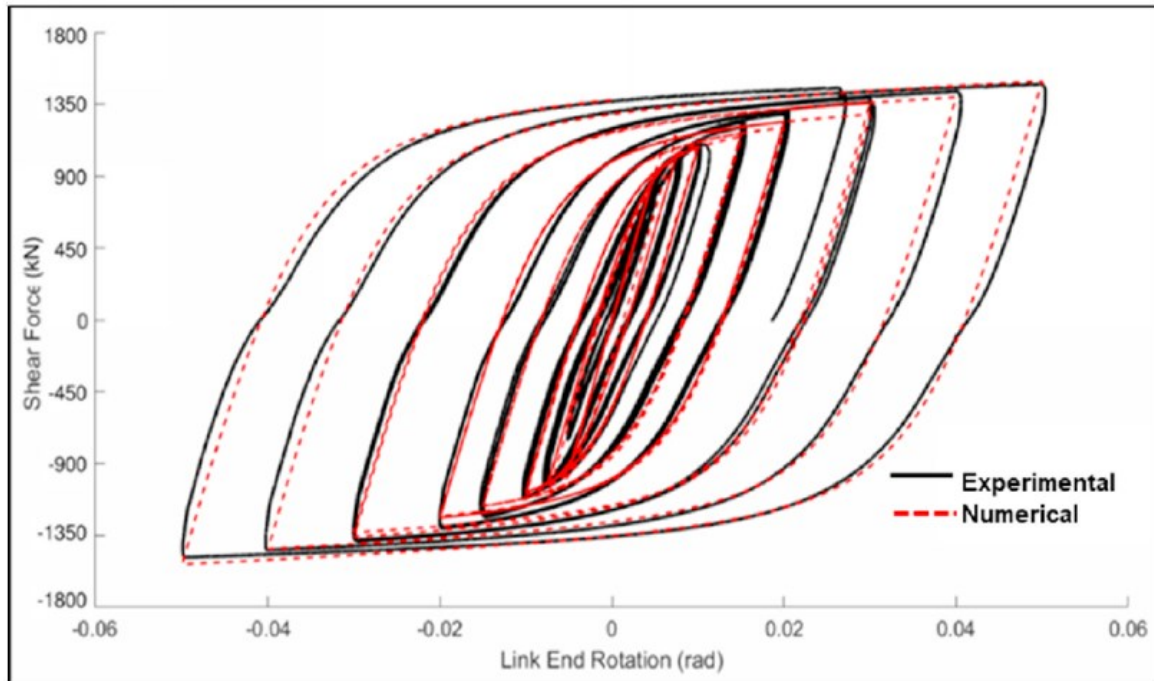


Figure 4-4 Calibration of the link against experimental data. (T.Y. Yang et al, 2020)

4.2 Modeling of beams out of the links and braces

The beams outside of the links and the braces were modeled using a non-linear beam-column element with a fiber section, taking into account the steel02 material. In fact, the method of combining subdivided elements with a specific number of fibers and integration points can influence the element's behaviour under cyclic loading (Figure 4-5). Several studies have been conducted in recent years to define and calibrate OpenSees modeling.

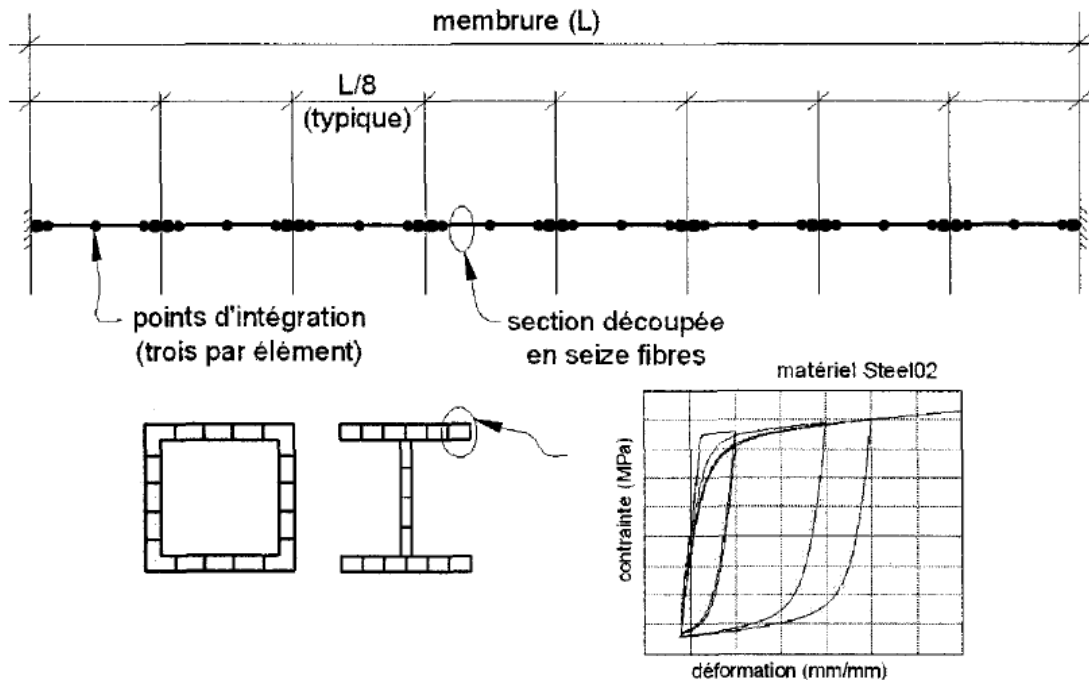


Figure 4-5 Modeling of nonlinear Beam Column element. (Rozon 2008)

In their research, Aguero et al. (2006) and Izvernari (2007) conducted a series of nonlinear temporal analyses using OpenSees to assess the seismic performance of steel "split-X" bracing. Within this study, Izvernari performed multiple parametric investigations aimed at determining the required level of accuracy for the inelastic column/beam elements within these frames. Key considerations included evaluating the effectiveness of the element employed, selecting the appropriate section type and its associated properties, deciding the number of integration points, the fiber numbers per element, and determining the material type and its relevant characteristics.

The study examines bracing members constructed from rectangular and square steel tubes. Initially, it assesses the efficiency and accuracy of force-based elements (FBE) and displacement-based elements (DBE). To model nonlinear geometric behaviour, a co-rotational framework is employed, and inelasticity is addressed by using fiber discretization of the cross-section in conjunction with a bi-linear or Giuffrè-Menegotto-Pinto hysteretic material response. A comparison is made between the FBE and DBE models across various combinations of factors such as the number of elements along the brace length, the number of integration points along the element length, the number of fibers used to define the cross-section, and the amplitude and number of displacement increments.

The results were compared to the experimental data, and the errors associated with the number of elements, fibers, and integration points are consolidated in Table 4-1. It is evident that force-based elements exhibit lower errors when compared to displacement-based elements.

Table 4-1 Mean quadratic error values (% P_y) (Aguero et al. 2006)

n_e	Displacement based elements			Force based elements		
	$n_i = 4$	$n_i = 5$	$n_i = 6$	$n_i = 4$	$n_i = 5$	$n_i = 6$
4	20.1	20.1	20.0	12.5	12.5	12.4
8	4.80	4.95	4.86	2.22	2.12	2.10
16	1.32	1.29	1.27	0.68	0.74	0.65
24	0.61	0.61	0.61	0.19	0.15	0.14
32	0.02	0.04	0.00	0.05	0.05	0.00

As we see in Figure 4-6 a and b error were reduced according to the increase of number of elements and the computational time increased

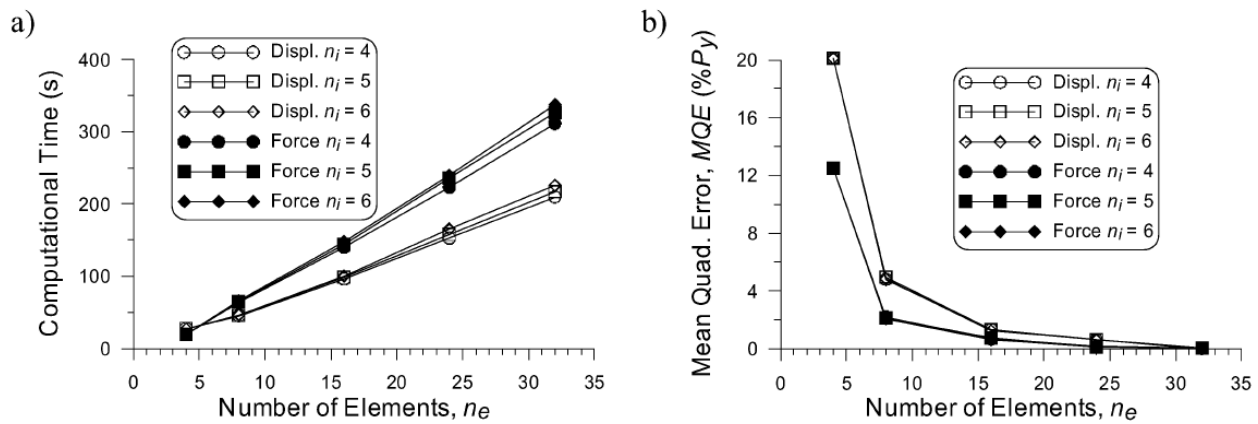


Figure 4-6 Computational time vs number of element, n_e and the errors. (Agüero et al, 2006)

The EBF model developed for the current study incorporated the suggestions made in this research regarding the quantity of fibers (16), integration points (8), and elements (8).

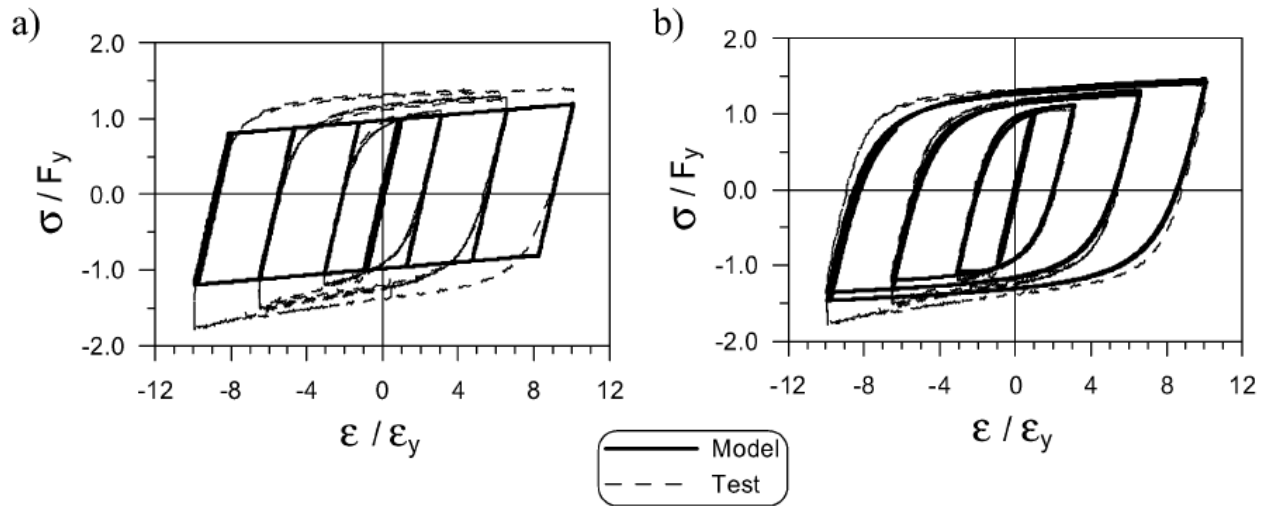


Figure 4-7 Experimental and numerical results of parametric assessment. (Agüero et al, 2006)

Damping

In their 1994 study, Ricles and Popov established that the incorporation of viscous damping in link elements can result in an excessive increase in link overstrength. Consequently, this leads to the introduction of excessively high forces into the remaining members of the EBFs. To address this issue, they recommend the utilization of non-proportional damping for EBFs as an alternative to the conventional Rayleigh damping approach. Non-proportional damping, in this context, involves the application of damping solely based on mass for the linked element, while Rayleigh damping continues to be employed for the other members. The specific definition of Rayleigh damping is outlined as follows:

$$C = c_0 M + c_1 K^* \quad (4.7)$$

The parameters c_0 and c_1 are related to mass and stiffness respectively and can be computed by knowing the damping value ξ at two modes. According to NEHRP ξ would be between 1% up to 5% match with building characteristics. In this research, a damping level equivalent to 3% of the critical damping in the first and third vibration modes was taken into consideration. This approach aligns with the findings of previous studies by Ricles and Popov (1994), Rossi and Lombardo (2007), and Koboevic and Redwood (1997). The values for c_0 and c_1 are computed using the following equations:

$$c_0 = \frac{2\xi(\omega_a\omega_b)}{\omega_a + \omega_b} \quad (4.8)$$

$$c_1 = \frac{2\xi}{\omega_a + \omega_b} \quad (4.9)$$

where ξ is the desired damping and ω_a and ω_b are the frequencies of the structure according to modes a and b.

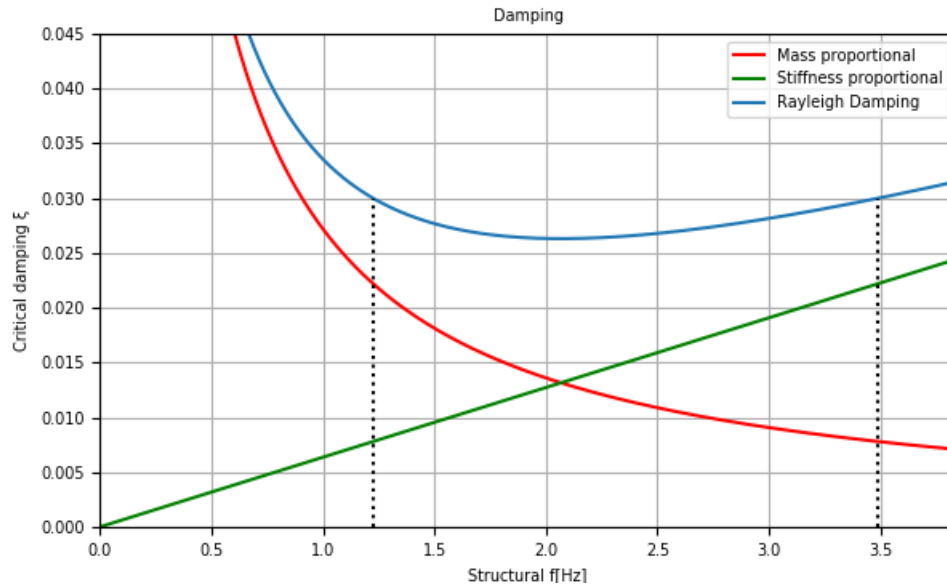


Figure 4-8 The damping curve defined by STKO software.

4.3 Soil-structure interaction modeling

In seismic conditions, a structure with shallow foundations may experience sliding, settling, and rocking motions. Even when the foundation's capacity is reached, the soil-foundation interface can absorb substantial seismic energy, leading to decreased forces transmitted to the superstructure. If the foundation's capacity is not utilized, these movements add flexibility to the system, potentially changing its response to earthquake ground motions. In both situations, it's essential to consider both temporary and permanent deformations. For effective performance-based seismic design, dependable numerical modeling tools are necessary to account for the advantages and drawbacks mentioned. The Winkler-based modeling framework is frequently used to replicate foundation behaviour during earthquakes. This model comprises an array of nonlinear springs, dashpots, and

gap elements, with the nonlinear spring characteristics calibrated against shallow foundation experiments. The model demonstrates a reasonable ability to predict the measured responses of footings in terms of moment, shear, settlement, and rotational demands.

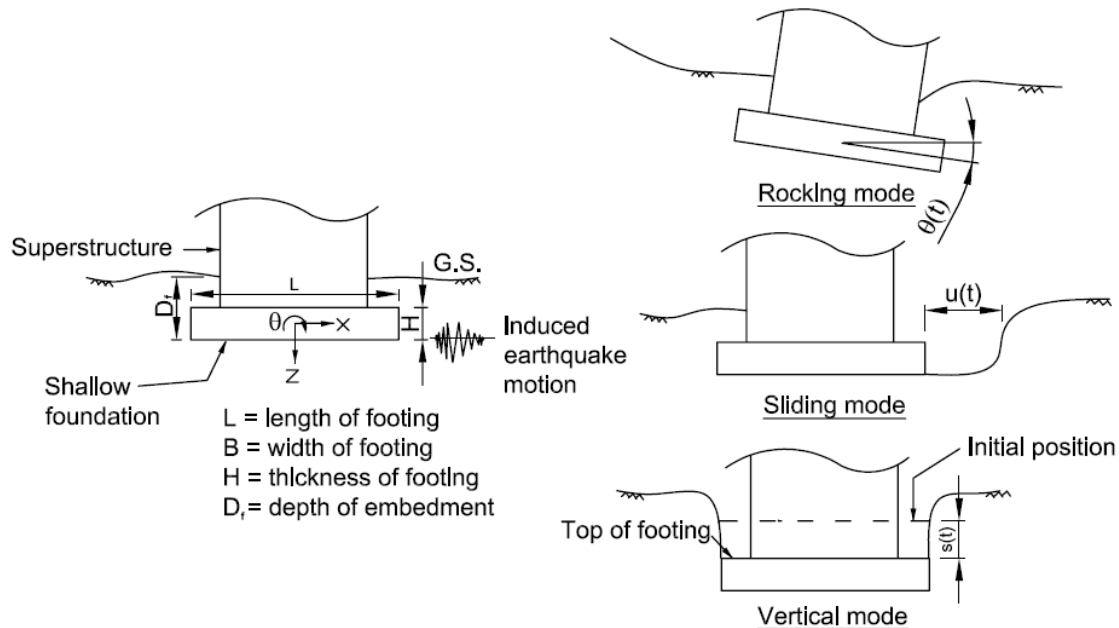


Figure 4-9 Different mode of foundation deformation. (Raychowdhury, 2008)

Several studies have employed Winkler springs with various characteristics to model shallow foundations and their interactions with structures. Chopra (1985) conducted an early study using nonlinear elastic-plastic Winkler springs. Nakaki (1987) used elastic, no-tension Winkler springs with viscous dampers for shear wall modeling and found that accounting for uplifting at the foundation base reduces ductility demands. Harden et al. (2005) used pile-calibrated nonlinear backbone curves for a Winkler-based model to simulate shallow strip footings supporting rocking-dominated shear wall buildings. Allotey and Nagga (2007) introduced a Winkler-based approach with multi-linear, no-tension backbone curves, comparing it with centrifuge tests conducted by Gajan (2003).

Raychowdhury (2008) did comprehensive research on Winkler-based model. This model can consider the behaviour of the soil-foundation system, taking into account inelastic soil behaviour (material nonlinearity) and uplift (geometric nonlinearity). It's important to differentiate between material nonlinearity, where a material may follow a nonlinear load-displacement path, and

inelastic behaviour, where it may not return along the same path due to factors like plastic deformation. The materials used in this study exhibit both nonlinearity and inelasticity, which can manifest in various ways, such as moment-rotation, shear-sliding, or axial-vertical displacement modes. Inelastic behaviour occurs due to the formation of gaps during cyclic loading. As a result, the model is capable of simulating phenomena like rocking, sliding, and permanent footing settlement, while also capturing the dissipation of energy through these behaviours.

In this model, one can introduce a flexible stiffness arrangement along the foundation's length to accommodate the increased response that may arise at the extremities of rigid footings under vertical loads. The BNWF model is designed to offer greater stiffness and a more closely spaced arrangement of vertical springs at the footing's ends, specifically to address rotational stiffness, as illustrated in Figure 4-10.

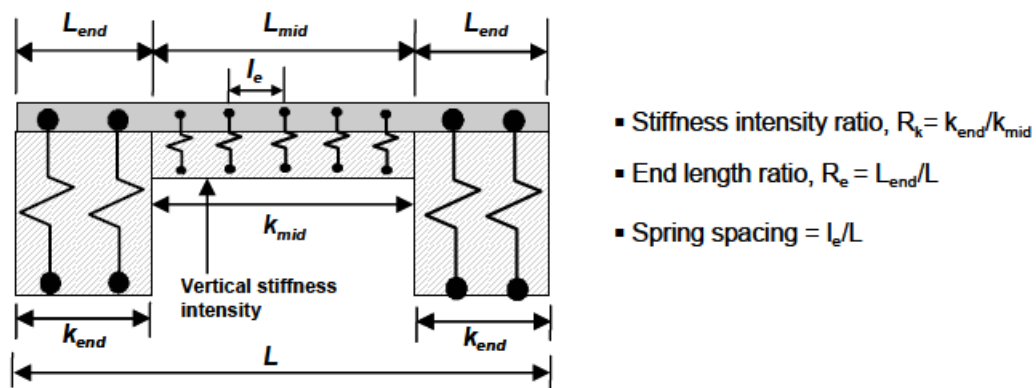


Figure 4-10 BNWF Mesh discretization with variable stiffness intensity (Harden et al., 2005)

4.3.1 Beam-on-Nonlinear-Winkler

This research employs the Beam-on-Nonlinear-Winkler-Foundation (BNWF) concept to simulate the behaviour of a shallow foundation under seismic loading in a two-dimensional (2D) context. It uses a mesh of nonlinear one-dimensional mechanistic spring elements to represent the connection between the structural foundation and the surrounding soil (free-field), addressing vertical and moment resistance. The constructed two-dimensional (2D) shallow foundation model is conceptualized as a flexible, elastic beam supported by multiple separate nonlinear Winkler springs as shown in Figure 4-11. The elastic beam is represented using one-dimensional beam-column elastic elements with two degrees of freedom (DOF) per node to account for vertical and rotational

loads and deformations. These individual nonlinear Winkler springs are independent of each other and are modeled as one-dimensional zero-length elements in the OpenSees framework, utilizing modified versions of the Qzsimple1 to represent nonlinearity.

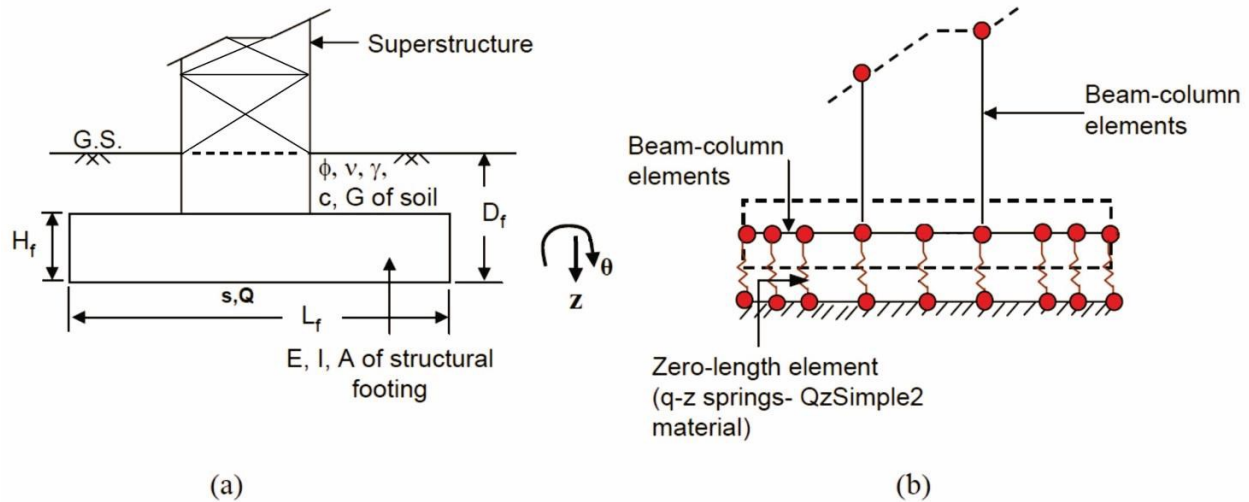


Figure 4-11 Winkler model. a) foundation geometry b) OpenSees modeling. (Raychowdhury 2008)

4.3.2 The QzSimple1

The QzSimple1 material exhibits an unsymmetric hysteretic response characterized by a backbone curve, with ultimate load in compression and reduced strength in tension to represent the soil's weak tension behaviour. It combines an elastic component to depict far-field behaviour with a plastic component for near-field permanent displacements. Additional gap components, like a drag and closure spring in series with the plastic parts, are included to simulate foundation uplifting behaviour (Figure 4-12). To address radiation damping, a dashpot is introduced on the far-field elastic component, with its viscous force proportional to the velocity developed in that component. The backbone curve features an elastic portion followed by an increasingly growing inelastic part (Figure 4-13).

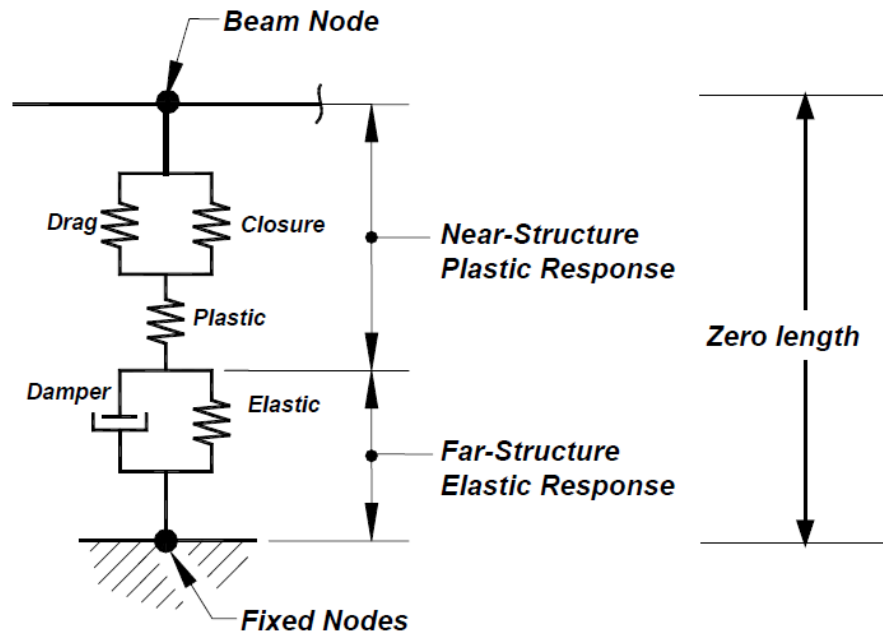


Figure 4-12 QzSimple1, PySimple1 and TzSimple1 material models used in conjunction with zero Length element (Raychowdhury 2008)

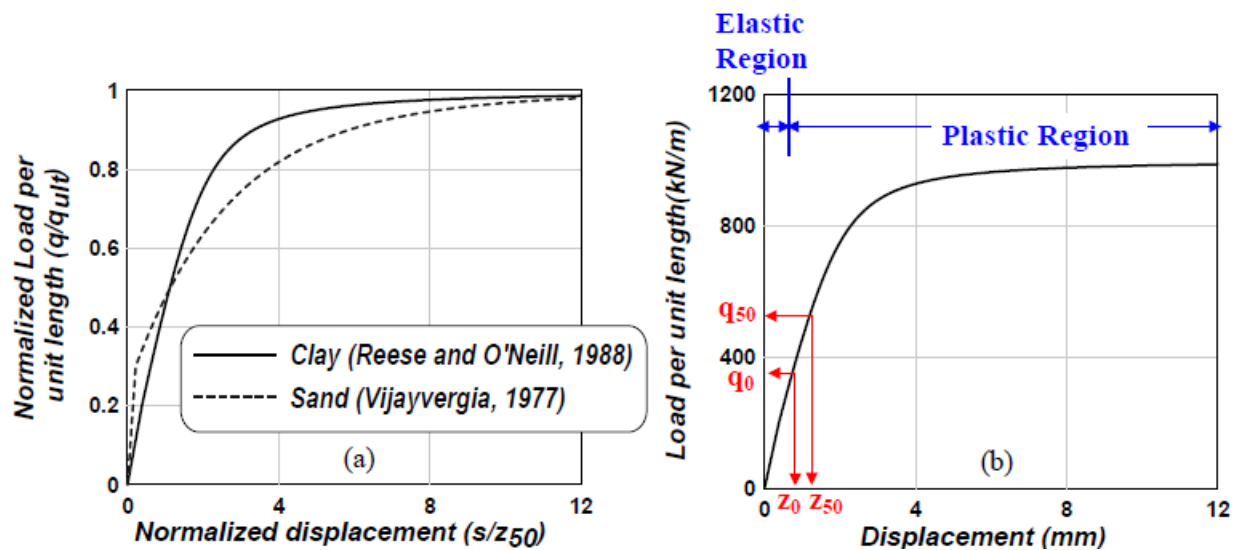


Figure 4-13 QzSimple1 backbone curve: (a) backbone curves used for soil-pile modeling and (b) general backbone curve (Raychowdhury 2008)

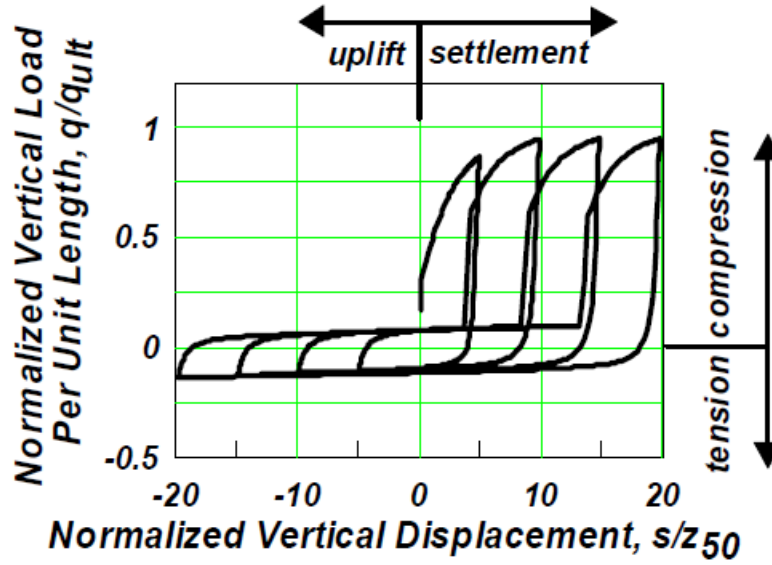


Figure 4-14 Cyclic response of QzSimple1 (Boulanger, 2000b)

The equations used to describe the QzSimple1 material model are as follows:

- Within the elastic part, the equation that defines the backbone curve is:

$$q = k_{in}z \quad (4.10)$$

- The extent of the elastic zone is determined by the subsequent equation:

$$q_o = C_r q_{ult} \quad (4.11)$$

where k_{in} = initial elastic stiffness (tangent), q = instantaneous load, z = instantaneous displacement, q_o = load at the yield point, and C_r = parameter controlling the range of the elastic portion.

- In the nonlinear (post-yield) part, the backbone curve is defined as follows:

$$q = q_{ult} - (q_{ult} - q_o) \left[\frac{cz_{50}}{cz_{50} + [z^p - z_o^p]} \right]^n \quad (4.12)$$

$$z_{50} = \frac{k_{fac}}{k_{in}} * q_{ult} \quad (4.13)$$

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.

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

z_0 = displacement at yielding point.

The spring's gap section consists of a combination of a closure spring and a drag spring. The closure part (q^c-z^g) is essentially a bilinear elastic spring, meaning it's stiff when compressed and highly flexible when stretched. The non-linear drag part (q^d-z^g) of the curve is governed by the following equation:

$$q_d = C_d q_{ult} - (C_d q_{ult} - q_0^d) \left[\frac{z_{50}}{z_{50} + 2[z^g - z_0^g]} \right] \quad (4.14)$$

Where:

q_d = drag force on the closure component.

$q_0^d = q^d$ at the start of the current loading cycle.

$z_0^g = z^g$ at the start of the current loading cycle.

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

The equations mentioned are suitable for both QzSimple1 and QzSimple2, two types of materials, with the only distinction being in the values of the parameters Cr, c , and n . These parameters for QzSimple2 were established in 2008 by Raychowdhury and Hutchinson through a regression analysis of laboratory tests on shallow foundations Table 4-2.

Table 4-2 QzSimple1 and QzSimple2 parameters

Soil type	parameter	QzSimple1	QzSimple2
Sand	c	12.3	9.29
	n	5.5	5.5
	Cr	0.30	0.36

Soil vertical stiffness

The vertical and lateral stiffness are essential parameters for QzSimple1 and can be determined using equations provided by Gazetas (1991a) or as recommended in ATC-40 (1996) Table 2-1. The foundation's vertical stiffness is indirectly considered through the differential movement of the vertical springs, which depends on how these springs are distributed along the footing's length Figure 4-15. Both vertical and lateral stiffness heavily rely on factors like the soil's shear modulus and the dimensions of the footing, including its length, width, and depth of embedment. The accuracy of estimating the shear modulus can significantly impact the stiffness of the springs and, consequently, the response of the entire foundation.

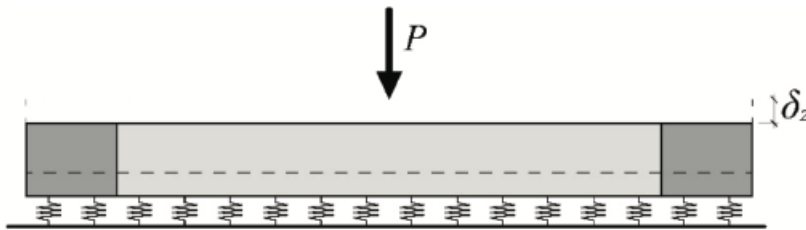


Figure 4-15 Distributed springs with vertical stiffness. (ASCE41 2013)

$$K'_z = \frac{GL}{1-v} \left[0.73 + 1.54 \left(\frac{B}{L} \right)^{0.75} \right] \quad (4.15)$$

$$\eta_z = \left[1 + \frac{D}{21 * B} \left(1 + \frac{1.3B}{L} \right) \right] \left[1 + 0.2 \left(\frac{A_w}{4BL} \right)^{\frac{2}{3}} \right] \quad (4.16)$$

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.

V = 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

η_z = Modification factor of vertical stiffness due to the embedment effect of the footing.

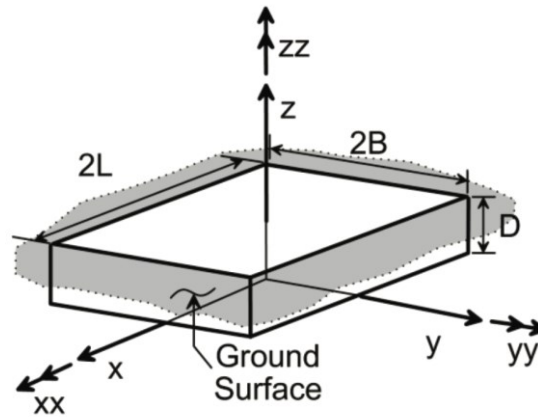


Figure 4-16 Footing geometric parameters. (NEHRP 2012)

End Length Ratio, R_e

In the model, we employ a varying stiffness distribution along the foundation's length to allocate vertical stiffness in a way that matches the rotational stiffness as described in Gazetas' work from 1991. The parameter R_e was defined as the proportion of the length of the higher stiffness region to the overall length of the footing.

$$R_e = \frac{L_{end}}{L} \quad (4.17)$$

ATC-40 (1996) recommends increasing stiffness at the ends of the foundation, defining an end region L_{end} as $B/6$ from each edge, irrespective of footing aspect ratio. However, Harden et al.

(2005) found that L_{end} varies with aspect ratio. For square footings (aspect ratio=1), their findings align with ATC-40 (1996) at approximately 16% of the footing's length (Figure 4-17).

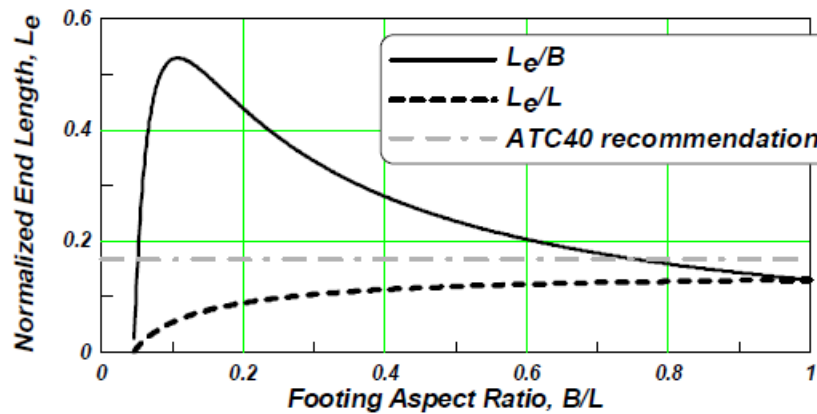


Figure 4-17 Normalized end length versus footing aspect ratio given by (Harden et al., 2005) along with ATC-40 (1996) recommended value. (Raychowdhury 2008)

It's important to observe that when R_e increases, the rotational stiffness of the foundation increases as well, leading to a greater demand for moments. However, a higher R_e , which means more rigid end springs, causes the foundation to experience greater permanent deformation because a stiffer spring yields earlier for a given input displacement.

Stiffness Intensity Ratio, R_k

Stiffness ratio R_k is defined as the ratio of the stiffness intensity at the end regions to that of the mid region.

$$R_k = \frac{k_{end}}{k_{mid}} \quad (4.18)$$

Harden et al. (2005) devised an equation to determine the end stiffness ratio by equating Gazetas' theoretical rotation stiffness equation with the rotational component achieved by increasing stiffness at the ends. This ratio varies with the footing's aspect ratio. Figure 4-18 compares this ratio, proposed by Harden et al. (2005), with ATC-40 (1996) recommendations. As the aspect ratio increases, so does the stiffness intensity ratio. Harden's ratio consistently remains lower than ATC-40's, indicating that ATC-40 suggests stiffer end regions compared to Harden et al.'s proposition based on Gazetas' rotation stiffness equation.

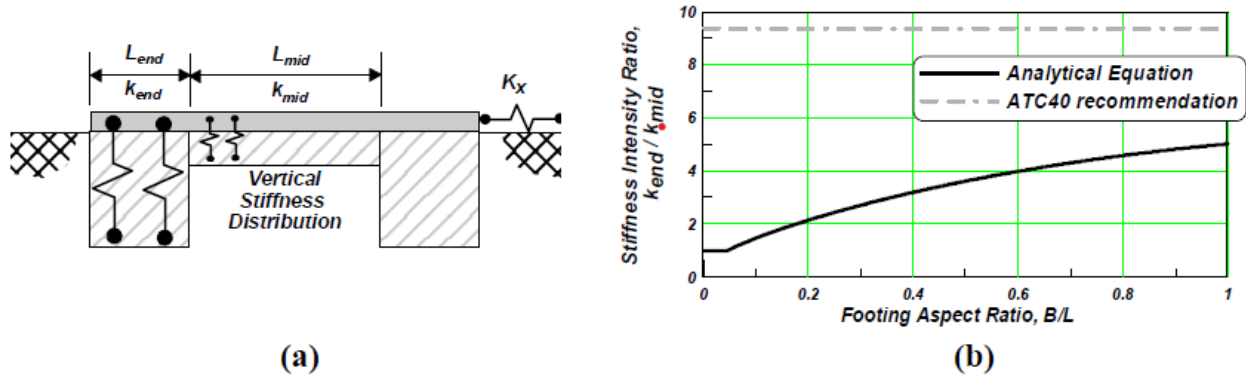


Figure 4-18 Increased end stiffness (a) spring distribution and (b) stiffness intensity ratio versus footing aspect ratio (Gajan 2007).

To address the issue of underestimated rotational stiffness, stiffer springs are applied to strips along the foundation's edge (with a length of R_e). When these are combined with the springs in the interior, the total rotational stiffness of the foundation is effectively restored. As illustrated in Figure 4-19, the stiffness of individual springs situated at the foundation's end regions is determined by their unitary vertical stiffness, their relevant tributary area, and the R_k factor. The equation used to calculate the R_k factor is obtained by comparing the moment generated by the springs when the foundation rotates by a unit amount and for a specified R_e ratio (usually chosen within the range of 0.3 to 0.5).

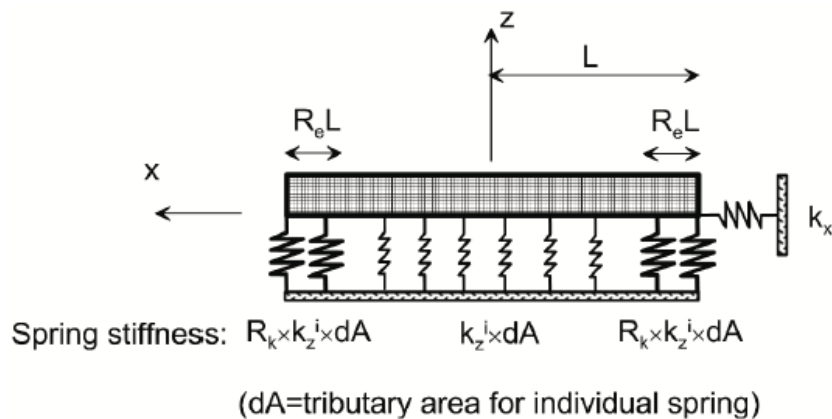


Figure 4-19 BNWF model with stiffer springs at the end regions. (NEHRP 2012)

R_k can be computed as follows:

$$k_z^i = \frac{k_z}{4BL} \quad (4.19)$$

$$k_{yy,surface} = \frac{G}{1-\nu} (I_y)^{0.75} \left[3 \left(\frac{L}{B} \right)^{0.15} \right] \quad (4.20)$$

$$\eta_{yy} = 1 + 0.92 \left(\frac{d_w}{B} \right)^{0.6} \left[1.5 + \left(\frac{d_w}{D} \right)^{1.9} \left(\frac{B}{L} \right)^{-0.6} \right] \quad (4.21)$$

$$k_{yy} = k_{yy,surface} * \eta_{yy} \quad (4.22)$$

$$R_{k,yy} = \frac{\left(\frac{3k_{yy}}{4k_z^i BL^3} \right) - (1 - R_e)^3}{1 - (1 - R_e)^3} \quad (4.23)$$

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

η_{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

In the BNWF model, the parameters dependent heavily on the analyst are the spacing of springs along the footing's length and the length of individual beam elements. The current OpenSees implementation maintains nearly uniform lengths for elements, regardless of their position. To enhance stiffness at the ends, the stiffness of springs in those regions is increased based on the stiffness intensity ratio. This adjustment aims to achieve higher stiffness intensity specifically in the end regions of the footing.

Raychowdhury (2008) demonstrated that augmenting the quantity of springs resulted in smoother moment-rotation curves (see Figure 4-20), as settlements and internal spring forces are more effectively dispersed across the footing's length. Conversely, reducing the number of springs leads to non-convergence and increased settlement in the model. The study also indicated that the model exhibited enhanced convergence in vertical displacements, settlements, and internal forces when the spring spacing constitutes approximately 4% of the foundation's length.

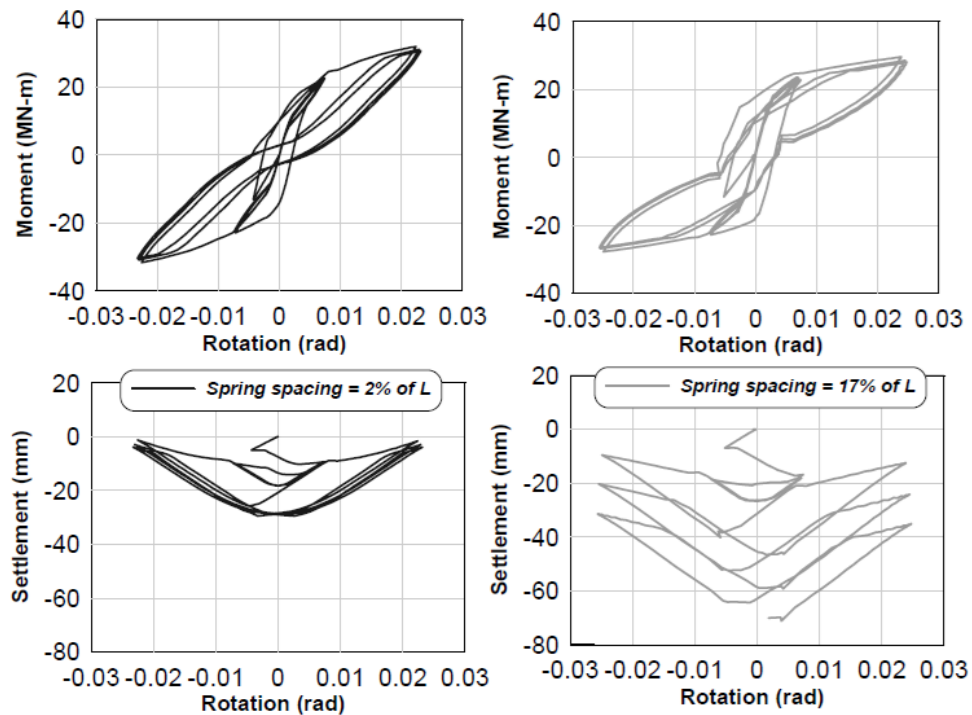


Figure 4-20 Effect of spring spacing on overall footing response: (a)–(b) moment versus rotation and (c)–(d) settlement versus rotation. (Raychowdhury 2008)

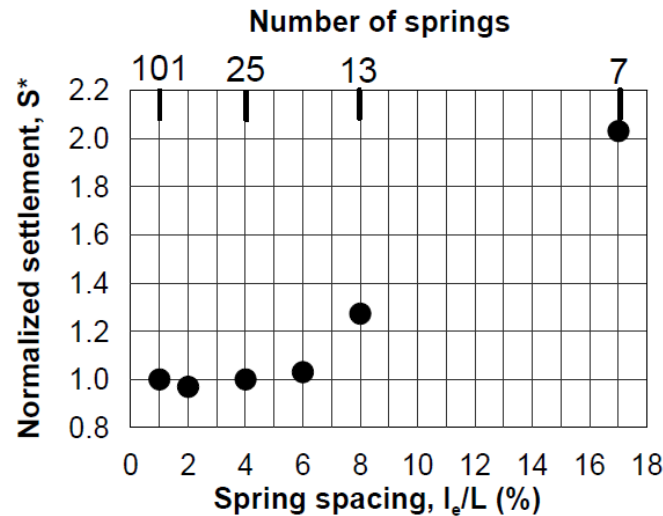


Figure 4-21 Effect of spring spacing on normalized settlement demand. (Raychowdhury 2008)

According to Raychowdhury (2008), to ensure improved convergence and smooth settlement, a uniform spring spacing of 400 mm is considered for all models. This spacing corresponds to approximately 2%-4% of the length of the foundation for different models, with the number of springs varying between 28 to 60 (Table 4-4) exceeding the minimum of 25 recommended by Raychowdhury (2008). The choice of uniform spring spacing, denoted as S_{sp} , involves increasing the stiffness of the springs within the L_{end} region of the footing by a rate of R_k . The R_k ratio is determined using equation 4.23 or by Figure 4-18 which was defined through Harden (2005) research. Following NEHRP's (2012) recommendations and aiming for consistency with Muruganathan's (2018) and Ray (2023) study, the R_e ratio remains constant across all scenarios at $0.40L_f$ of the total length of the footing (0.20 for each L_{end}).

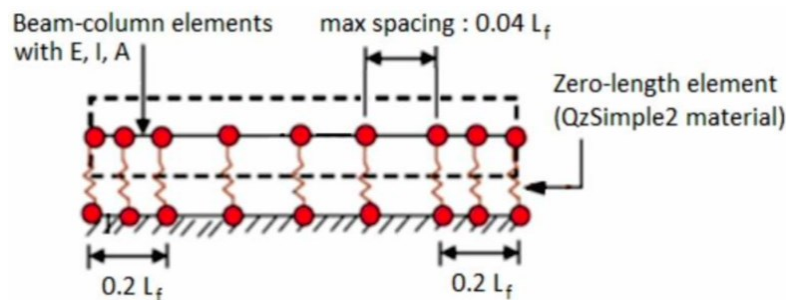


Figure 4-22 System Soil-Column-Foundation. OpenSees modeling of the system. (Raychowdhury and Hutchinson 2008)

Table 4-3 Foundation vertical and rotational stiffness for different scenario in Vancouver.

Storey	Site Class	M_{design}	FOOTING DIMENSIONS			qult (kPa)	Kz (kN/mm)	Kyy (MNm/rad)
			h (m)	b (m)	t (m)			
3	C	M_{prob}	11.2	4.5	1.3	3000	2251	50434
		$M_{\text{RdRo}=6}$	11.2	2	1.3	3000	1722	31004
	E	M_{prob}	16.8	6	1.7	400	646	31720
		$M_{\text{RdRo}=6}$	16.8	5	1.7	400	605	28433
8	C	M_{prob}	16	6	1.5	3000	3132	141076
		$M_{\text{RdRo}=6}$	16	3.5	1.5	3000	2608	102095
	E	M_{prob}	24	12	2	400	1053	113167
		$M_{\text{RdRo}=6}$	24	9	2	400	940	95226

Table 4-4 Soil spring properties for different scenarios in Vancouver

Storey	Site Class	M_{design}	Spring space (mm)	N (Qty.)	L end (mm)	R_k	R_e	Z50 mid (mm)	Z50 end (mm)
3	C	M_{prob}	400	28	2240	2.46	0.40	81.3	33.1
		$M_{\text{RdRo}=6}$	400	28	2240	1.92	0.40	47.2	24.6
	E	M_{prob}	400	42	3360	2.39	0.40	75.5	31.6
		$M_{\text{RdRo}=6}$	400	42	3360	2.27	0.40	67.2	29.6
8	C	M_{prob}	400	40	3200	2.42	0.40	111.3	46.0
		$M_{\text{RdRo}=6}$	400	40	3200	2.06	0.40	77.9	37.7
	E	M_{prob}	400	60	4800	2.58	0.40	132.3	51.3
		$M_{\text{RdRo}=6}$	400	60	4800	2.42	0.40	111.3	46.0

Table 4-5 Foundation vertical and rotational stiffness for different scenario in Montreal.

Storey	Site Class	M_{design}	FOOTING DIMENSIONS			qult (kPa)	Kz (kN/mm)	Kyy (MNm/rad)
			h (m)	b (m)	t (m)			
3	C	M_{prob}	11.2	4.5	1.3	3000	2251	50434
		$M_{RdRo=6}$	11.2	2	1.3	3000	1722	31004
	E	M_{prob}	16	6	1.5	400	626	28215
		$M_{RdRo=6}$	16	5	1.5	400	586	25291
8	C	M_{prob}	16	5.5	1.5	3000	3032	133900
		$M_{RdRo=6}$	16	3	1.5	3000	2493	93075
	E	M_{prob}	21.6	11	2	400	955	83412
		$M_{RdRo=6}$	21.6	9.5	2	400	900	76388

Table 4-6 Soil spring properties for different scenarios in Montreal.

Storey	Site Class	M_{design}	Spring space (mm)	N (Qty.)	L end (mm)	Rk	Re	Z50 mid (mm)	Z50 end (mm)
3	C	M_{prob}	400	28	2240	2.46	0.40	81.3	33.1
		$M_{RdRo=6}$	400	28	2240	1.92	0.40	47.2	24.6
	E	M_{prob}	400	40	3200	2.42	0.40	74.2	30.7
		$M_{RdRo=6}$	400	40	3200	2.30	0.40	66.1	28.7
8	C	M_{prob}	400	40	3200	2.36	0.40	105.3	44.5
		$M_{RdRo=6}$	400	40	3200	1.96	0.40	69.9	35.7
	E	M_{prob}	400	54	4320	2.59	0.40	120.4	46.5
		$M_{RdRo=6}$	400	54	4320	2.51	0.40	110.4	44.0

4.4 Ground Motions

Predicting and modeling accelerograms constitutes one of the most crucial aspects of conducting an NLTH analysis. As nonlinear dynamic analysis progresses and integrates into performance-based design, it becomes vital to explore the effect of selecting accelerograms on predicting structural responses. Given that accelerograms recorded at various locations originate from entirely random processes that are nearly impossible to replicate, significant efforts have been focused on choosing and optimizing genuine accelerograms. The goal is to make sure that these depictions faithfully represent future seismic events and reduce differences in structural reactions.

NBC Seismic Hazard Model

Before going through the selection and scaling the ground motions it good to know how the target spectrum for different zone of Canada was defined. The NBC 2015 seismic hazard model was crafted using the Cornell–McGuire probabilistic approach, applied to fault and earthquake sources in Canada and nearby areas. This model carefully addresses uncertainties in critical input factors like seismic activity rates, maximum magnitudes, focal depth, ground motion relationships, and source zone models. It identifies two types of uncertainty: one stemming from the inherent variability in future events (aleatory), and the other from differences in modeling assumptions, unknown parameters, and extending beyond observed data (epistemic). Accurate assessments of these uncertainties enable the calculation of ground motions for a specific probability of occurrence with a desired confidence level, like mean, median, or median with one standard deviation.

The seismic hazard data in the NBC 2005 and NBC 2010 relied on two models: the H model, using historical data, and the R model, based on larger regional zones influenced by geological and seismotectonic factors. In contrast, the NBC 2015 seismic hazard model Adams et al., combined updated historical and regional models probabilistically for northeastern Canada. For southeastern Canada, a mix of historical and regional elements was added to updated models, creating three combined probabilistic models. Western Canada used a single set of source models but with variations in shape, especially concerning the proximity of the Cascadia subduction zone to southwestern British Columbia. These models were updated to reflect new information and recalculate their seismic parameters. The NBC 2005 model featured the Queen Charlotte fault as a fault source and a determined approach to the Cascadia subduction zone. In contrast, the NBC 2015 model included various fault sources like low-angle subduction thrusts, an updated offshore Queen

Charlotte fault source, and active fault sources in the Alaska–Yukon region. Low-angle subduction thrust faults of different depths were used to model various parts of the Cascadia subduction zone and the Haida Gwaii thrust, focusing on the potential for seismic activity down the line. Some faults in Alaska and Washington weren't explicitly included in the 2015 model, as their seismic hazard contributions in Canada were sufficiently represented by the sources used in the model. These models primarily catered to Canada's more active seismic regions, while about half of the country, considered tectonically stable, lacked enough seismic activity to define reliable source zones. Considering the potential for infrequent but significant earthquakes in these stable yet less seismically active regions, it's crucial to ensure accurate estimates of seismic hazard. Updated considerations from Adams and Halchuk's work in NBC 2010 are applied to evaluate seismic activity levels in these stable regions, aiming to provide reliable seismic hazard estimates even in areas with fewer seismic events.

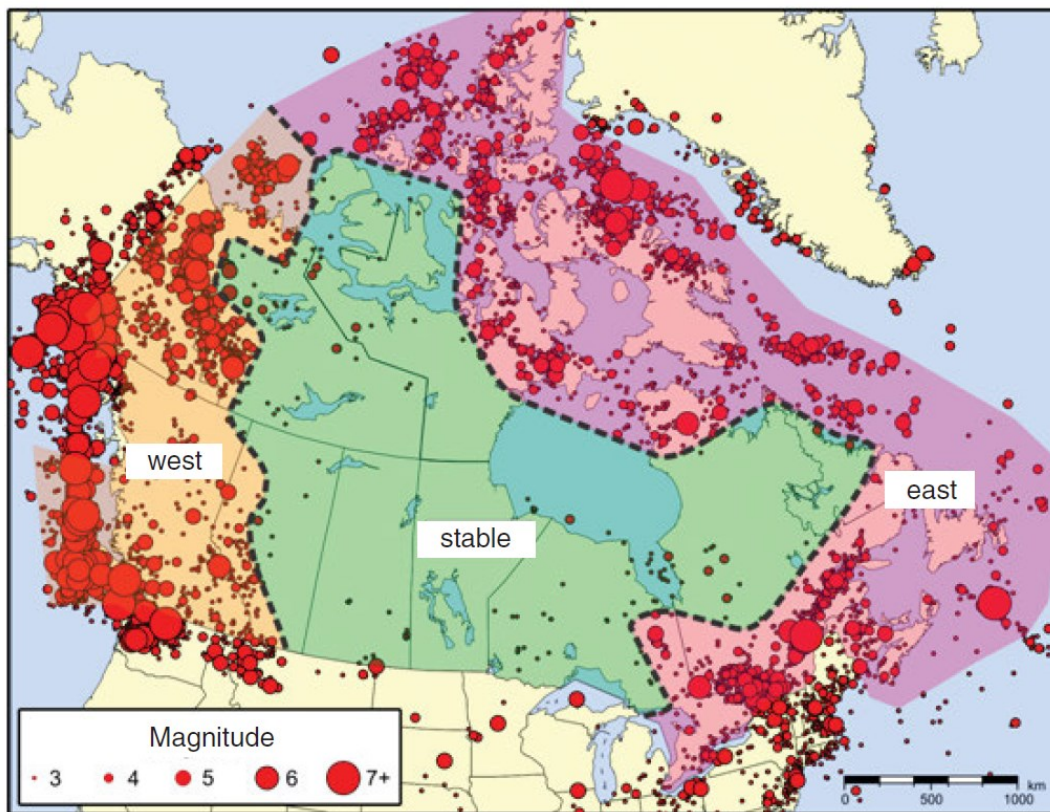


Figure 4-23 Map of seismicity in Canada to 2010, delineating the eastern and western seismic regions and the low seismicity central continental region. (user's guide NBC 2015)

4.4.1 Selection and Scaling of Ground Motions

Sets of accelerograms were carefully chosen based on specific combinations of intensity (M) and hypo central distance (R), which are key parameters for assessing seismic risk decomposition. When selecting the suitable accelerograms for a building, one must carefully consider several factors. These include the building's characteristics and the seismic source's attributes, such as earthquake magnitude, fault mechanism (e.g., direct, reverse), focal depth, and distance from the building to the earthquake source. Additionally, considerations should be made regarding the geological conditions of the area, maximum acceleration levels, frequency content, earthquake duration, and energy. This study focused on two locations, Montreal and Vancouver, representing seismic activity typical of Eastern and Western Canada, respectively. Historical seismic events were selected if they matched the appropriate M-R combination for the respective city. Additionally, artificial accelerograms were included to compensate for gaps in historical data availability in Eastern Canada.

For this study, the structural design followed NBCC 2020 guidelines. The code incorporates the concept of the uniform hazard spectrum (UHS), which is specific to each city in the country. This spectrum illustrates the spectral acceleration concerning the fundamental period of a structure. The UHS was defined using a 2% probability of exceedance over a 50-year period. Additionally, there is a notable increase in the target response spectrum in CNBC 2020 compared to the CNBC 2015 edition.

Period Range which impacts the structure's inelastic seismic response

The process involves selecting and scaling ground motion time histories to align with a target response spectrum derived from the design spectrum $S(T)$ for a particular period range of interest. This design spectrum is determined through a probabilistic seismic hazard analysis that calculates the Uniform Hazard Spectrum (UHS) for a specified probability of exceeding certain seismic conditions.

The definition of the target spectrum relies on the reference to $S(T)$ and must include a range of periods, T_R , that significantly impact the structure's inelastic seismic response in translational directions and/or torsion. The lower limit of this period range, T_{min} , is set to match the period of the highest vibration mode needed to account for at least 90% of the structure's mass ($T_{90\%}$). T_{min}

should not be more than 0.2 times the period of the structure's fundamental mode, T_1 . For the upper limit, $T_{\max}$ is set to twice the period of T_1 ($2.0 T_1$). Additionally, $T_{\max}$ should be at least 1.5 seconds to ensure that the ground motion records accurately reflect the seismic demands for stiff structures, covering the period range where most of the energy from typical earthquakes is found. In conclusion, the period range Figure 4-24 designated for selecting and scaling ground motion is as follows: $[T_{\min}, 1.5 \text{ s} \leq T_{\max} \leq 2.0 T_1]$ (Atkinson and Tremblay, 2015).

According to the different structure's period which came through static and modal analysis in chapter 3 and with considering the requirement of this section, period range for different scenario were defined as following

Table 4-7.

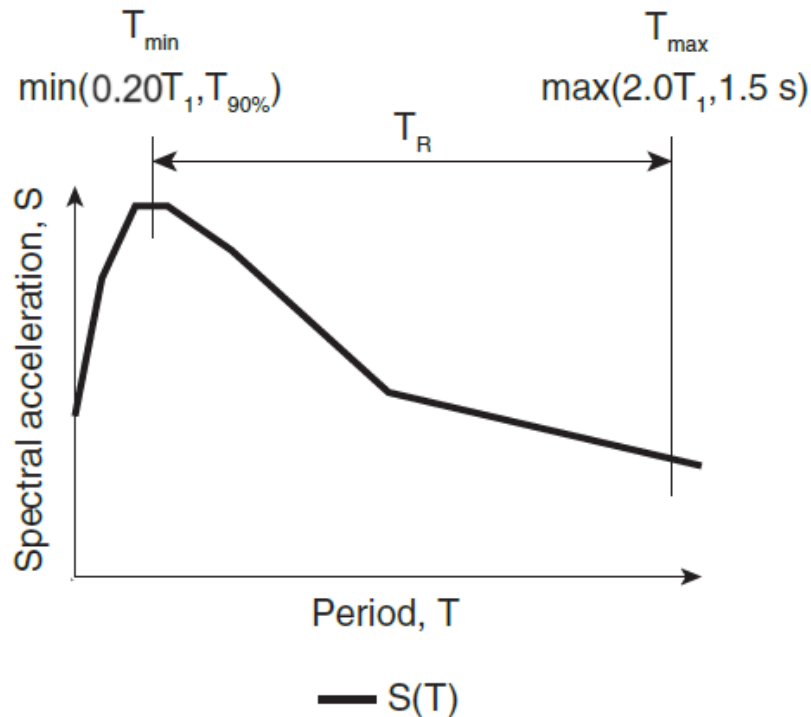


Figure 4-24 period range T_R (user's guide NBC 2015)

Table 4-7 The period range of interest for 3-and 8-storey structure in Montreal and Vancouver were computed.

Scenarios	Code	Location	Storey	Site	Foundation	T1 (s)	T _{min} (s)	T _{max} (s)
SC1	NBC2020	MTL	3	C	FIX	0.69	0.14	1.50
SC2				E	FIX	0.61	0.12	1.50
SC3			8	C	FIX	1.81	0.36	3.62
SC4				E	FIX	1.63	0.33	3.25
SC5		VCR	3	C	FIX	0.54	0.11	1.50
SC6				E	FIX	0.45	0.09	1.50
SC7			8	C	FIX	1.33	0.27	2.66
SC8				E	FIX	0.98	0.20	1.96

4.4.2 Deaggregation of mean hazard for Montreal and Vancouver

To gain insights into the seismic hazard results for certain cities, deaggregation, was performed which involves breaking down the earthquake contributions based on their distance and magnitude. This analysis covers a range of probabilities and spectral accelerations (S_a) from 0.1 to 10.0 seconds, focusing on Vancouver and Montréal, the two major urban centers in Canada facing the highest risk.

Deaggregations were conducted on the target response spectrum for both Vancouver and Montreal Table 4-8. This process involved analyzing a range of accelerations (S_a) from 0.1 to 10.0 seconds. The results of this analysis assisted in identifying potential earthquake sources and the predominant magnitude-distance (M-R) scenarios unique to these two cities. As a result, these findings guided the selection of appropriate ground motion time histories and determined the minimum scaling prerequisites essential for seismic analysis and design.

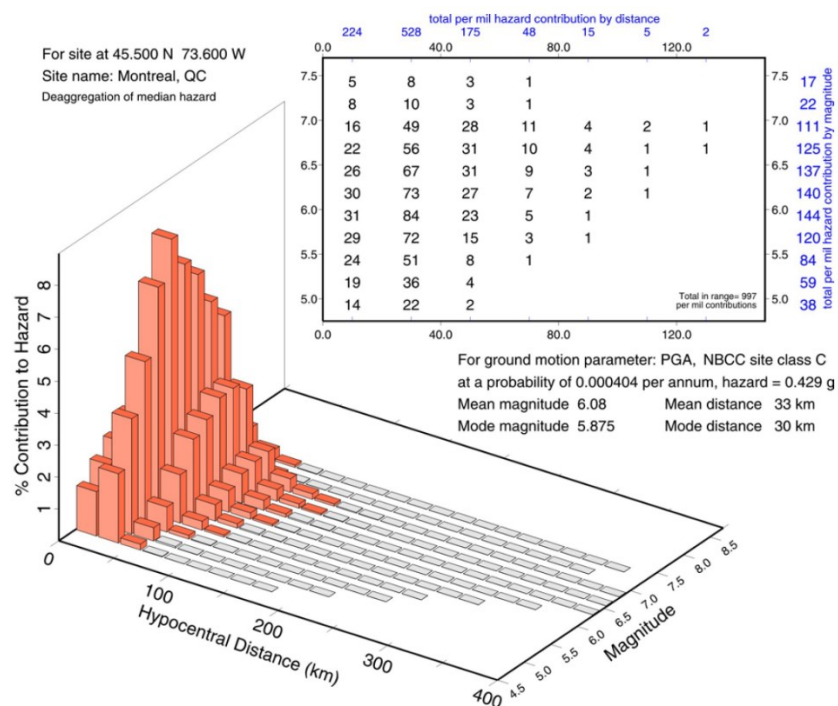


Figure 4-25 Deaggregation of Montreal PGA for a probability of 2%/50 years. Numerical values in the upper right box allow for an analytical examination (Halchuk et al., 2007)

Notably, larger earthquakes that are farther away have a more significant impact on the hazard over longer periods, compared to shorter periods (Figure 4-26). These deaggregations permit to make well-informed decisions regarding scenario events and selecting representative time histories for seismic design purposes. (Stephen Halchuk et al., 2019). As an example, the deaggregation for Vancouver indicates that the in-slab type of earthquake contributes most significantly to the seismic hazard in this region.

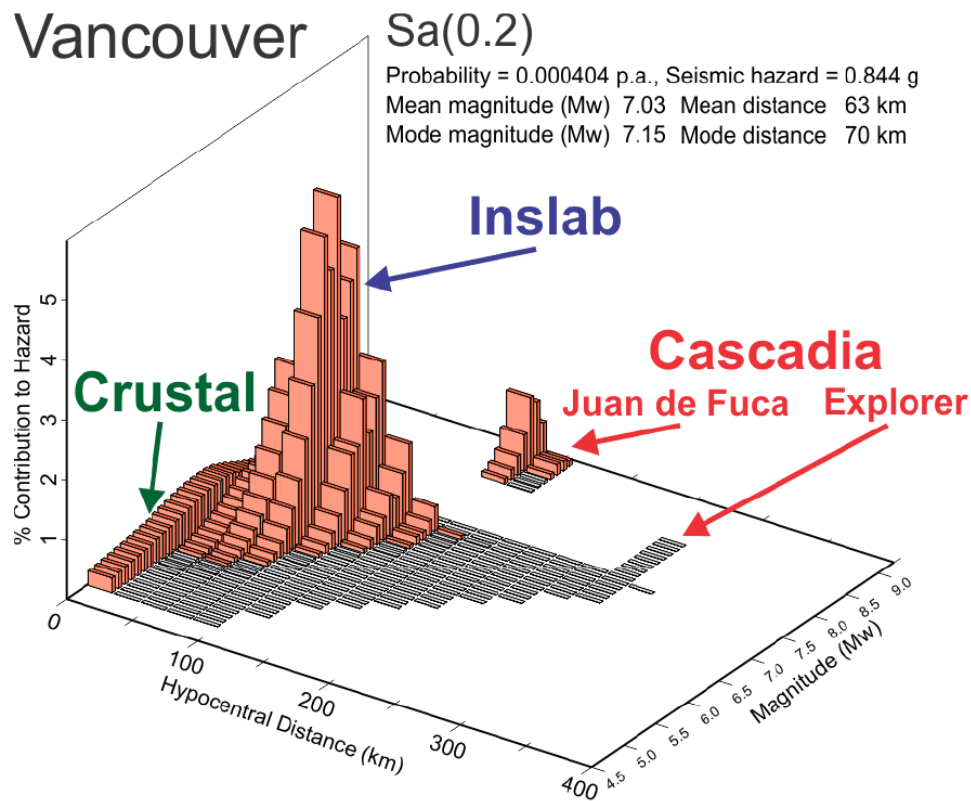


Figure 4-26 Deaggregation of Vancouver for a probability of 2%/50 years (NBCC2015) for $S_a(0.2)$. Contributions from various earthquake sources are indicated in the figure. (Stephen Halchuk et al. 2019)

Table 4-8 Deaggregation of mean hazard for Montreal and Vancouver

Deaggregation of mean hazard for Montreal and Vancouver				
Mean magnitude (Mw) and Mean distance (Soil C)				
Spectral	Montreal		Vancouver	
Period (s)	Mw	km	Mw	km
0.1	6.40	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
3	7.09	78	8.40	119.0
5	7.17	88	8.68	131
10	7.25	111	8.74	134
probability of 0.000404 per annum				

Table 4-9 The average magnitude-distance (M-R) value resulting from the period range for two scenarios.

Deaggregation summary	Earthquake type	Period (s)	Montreal		Vancouver	
		range	Mw	kM	Mw	kM
scenario 1	Crustal, in-slab (near field)	0.1 - 0.5	6.55	31	7.10	66
scenario 2	Interface (Far field)	1.0 - 5.0	7.03	71	8.10	106

To facilitate the selection process, it is essential to define suitable magnitude and distance ranges for each M-R scenario Table 4-9. For example, in the case of an M6.7-R25 km scenario, the ground motion time-histories could be chosen from earthquakes with magnitudes ranging between 6.5 and 7.0, and recordings made at distances spanning from 20 to 30 km (Halchuck et al., 2014).

4.4.3 Selection of accelerogram for Vancouver site class E and C

Different databases were verified to find proper accelerograms which could be matched with our project M-R scenario. Databases are Peer (Crustal, in-slab), New Zealand (Interface, Crustal, in-slab), Japan (Interface) and Mexico (Interface). The total of 274 accelerograms were selected according to the M-R scenario for Vancouver.

Table 4-10 selected ground motions.

Vancouver	Peer	New Zealand	Japan	Mexico	Total
M7-R60 soil C	35	18	0	0	53
M7-R60 soil E	27	23	0	0	50
M8-R100 soil C	15	11	57	2	85
M8-R100 soil E	15	21	49	1	86
Total	92	73	106	3	274

The NBCC Guidelines mandate using a collection of at least eleven ground motion time histories for dynamic analysis. When dealing with two or more seismic M-R scenarios, a minimum of five ground motions must be selected for each scenario, termed as "suites." For instance, if two M-R scenarios are considered, one would need a suite of five ground motions and another suite with six ground motions. These prescribed minimum numbers strike a balance between achieving statistically reliable estimates of mean structural responses and ensuring the computational effort remains practical (Haselton et al., 2004).

To ensure the most suitable ground motion candidates are chosen for a specific M-R scenario, it may be necessary to refine the selection process. The final selection involves favoring motions with spectral shapes that closely align with the target response spectrum within the corresponding period range. Each ground motion needs to be individually adjusted so that, on average, its response

spectrum $S_g(T)$ matches or surpasses the target response spectrum, $S_T(T)$, within the suitable period range specified for each scenario.

Furthermore, all records within each set of time histories should be adjusted using a common factor. This factor ensures that the average response spectrum of the set, doesn't drop more than 10% below the target spectrum, $S_T(T)$, for any specified period outlined in Section 4.4., within the appropriate scenario-specific period range, TRS.

It's important to be cautious when extremely low or high scaling factors $S_T(T)/S_g(T)$ occur (e.g., less than 0.5 or greater than 2.0). Such cases might indicate that the ground motion isn't compatible with the source mechanisms or seismic hazard level being considered. This situation could necessitate a review of ground motion selection.

Final selection for groundmotions

Among the total of 274 ground motions, taking into account all types of earthquakes such as Crustal, in-slab, and Interface, 15 accelerations with less deviation from the target spectrum were chosen for Vancouver soil types C and E. This implies that a total of 30 accelerograms were selected.

Table 4-11- Selected ground motion for Vancouver site class C

Record ID	Record	EVENT TYPE	Earthquake Name	Year	Magnitude	R (km)
1	RSN801_LOMAP_SJTE315.AT2	Crustal	"Loma Prieta"	1989	6.93	14.18
2	RSN265_VICT_CPE045.AT2	Crustal	"Victoria_ Mexico"	1980	6.33	13.8
3	RSN2627_CHICHI.03_TCU076E.AT2	Crustal	"Chi-Chi_ Taiwan-03"	1999	6.20	13.04
4	RSN4850_CHUETSU_65013EW.AT2	Crustal	"Chuetsu-oki_ Japan"	2007	6.80	13.68
5	RSN3748_CAPEMEND_FFS270.AT2	Crustal	"Cape Mendocino"	1992	7.01	16.64
6	RSN755_LOMAP_CYC285.AT2	Crustal	"Loma Prieta"	1989	6.93	19.97
7	103I56ol_y0a_Scaled.acc	Inslab	USA, Olympia	1949	6.70	50
8	103I56ol_y0b_Scaled.acc	Inslab	USA, Olympia	1949	6.70	50
9	0725a_a_Scaled.acc	Inslab	USA, Nisqually	2001	6.93	20.34
10	MYG0111103111446.EW	Interface	TOHOKU 2011	2011	9.00	120
11	FKS0071103111446.EW	Interface	TOHOKU 2011	2011	9.00	184
12	KSRH050309260450.NS2	Interface	TOKACHI-OKI 2003	2003	8.00	164
13	HKD1110309260450.EW	Interface	TOKACHI-OKI 2003	2003	8.00	81
14	HKD1110309260450.NS	Interface	TOKACHI-OKI 2003	2003	8.00	81
15	HKD1090309260450.EW	Interface	TOKACHI-OKI 2003	2003	8.00	117

Table 4-12 Selected ground motion for Vancouver site class E.

No	Record	EVENT TYPE	Earthquake Name	Year	Magnitude	Mechanism	Rjb (km)
1	RSN728_SUPER.B_B-WSM180.AT2	Crustal	"Superstition Hills-02"	1987	6.54	strike slip	13.03
2	RSN6890_DARFIELD_CMHSN10E.AT2	Crustal	"Darfield_ New Zealand"	2010	7.00	strike slip	17.64
3	RSN6959_DARFIELD_REHSN02E.AT2	Crustal	"Darfield_ New Zealand"	2010	7.00	strike slip	19.48
4	RSN758_LOMAP_EMY350.AT2	Crustal	"Loma Prieta"	1989	6.93	Reverse Oblique	76.87
5	RSN759_LOMAP_A01000.AT2	Crustal	"Loma Prieta"	1989	6.93	Reverse Oblique	43.77
6	RSN6959_DARFIELD_REHSS88E.AT2	Crustal	"Darfield_ New Zealand"	2010	7.00	strike slip	19.48
7	2147b_a_Scaled.acc	Inslab	--	--	--	--	--
8	1412c_a_Scaled.acc	Inslab	--	--	--	--	--
9	MYG0041103111446.EW	Interface	TOHOKU 2011	2011	9.00	--	175
10	KKWH080309260450.NS1	Interface	TOKACHI-OKI 2003	2003	8.00	--	182
11	HKD1260309260450.NS	Interface	TOKACHI-OKI 2003	2003	8.00	--	198
12	KKWH080309260450.NS2	Interface	TOKACHI-OKI 2003	2003	8.00	--	182
13	HKD1080309260450.EW	Interface	TOKACHI-OKI 2003	2003	8.00	--	136
14	HKD1000309260450.NS	Interface	TOKACHI-OKI 2003	2003	8.00	--	84
15	HKD1260309260450.EW	Interface	TOKACHI-OKI 2003	2003	8.00	--	198

4.4.4 Scaling of Ground Motion Records

Based on the NBCC guidelines, it is required to scale each ground motion so that its response spectrum $S_g(T)$ is generally equal to or exceeds the target spectrum $S_T(T)$ within the appropriate period range. Furthermore, the mean response spectrum of each scenario-specific set of time histories should not deviate more than 10% below $S_T(T)$ over the corresponding period range.

4.4.4.1 First method for Scaling the ground motion

The scaling process was conducted separately for 3-and 8-storey buildings due to their distinct period ranges. During the initial scaling for the Vancouver 3-storey building with soil type C, all aspects aligned with the specifications outlined in the user's guide NBC 2015 Figure 4-28. However, for the Vancouver 8-storey building with soil type C, the mean response spectrum deviated by more than 10% below $S_T(T)$ Figure 4-29. Consequently, a second scaling factor of 1.15 was applied to all the accelerograms to rectify this discrepancy Figure 4-30.

Table 4-13 Scale factor for Vancouver site class C, 3 and 8 storey buildings.

No	Record ID	Scale factor (3 storey)	Scale factor (8 storey)	Record	EVENT TYPE
1	378	1.97	3.22	RSN801_LOMAP_SJTE315.AT2	Crustal
2	343	0.99	2.09	RSN265_VICT_CPE045.AT2	Crustal
3	425	0.96	1.73	RSN2627_CHICHI.03_TCU076E.AT2	Crustal
4	488	0.93	1.40	RSN4850_CHUETSU_65013EW.AT2	Crustal
5	439	0.91	0.95	RSN3748_CAPEMEND_FFS270.AT2	Crustal
6	374	1.29	1.77	RSN755_LOMAP_CYC285.AT2	Crustal
7	RBT1	1.37	1.82	103I56ol_y0a_Scaled.acc	in-slab
8	RBT2	1.65	2.04	103I56ol_y0b_Scaled.acc	in-slab
9	RBT3	1.82	2.39	0725a_a_Scaled.acc	in-slab
10	JP83	0.68	1.02	MYG0111103111446.EW	Interface
11	JP9	0.77	1.17	FKS0071103111446.EW	Interface
12	JP264	1.71	2.18	KSRH050309260450.NS2	Interface
13	JP231	1.07	1.37	HKD1110309260450.EW	Interface
14	JP232	1.04	1.19	HKD1110309260450.NS	Interface
15	JP227	0.85	1.67	HKD1090309260450.EW	Interface

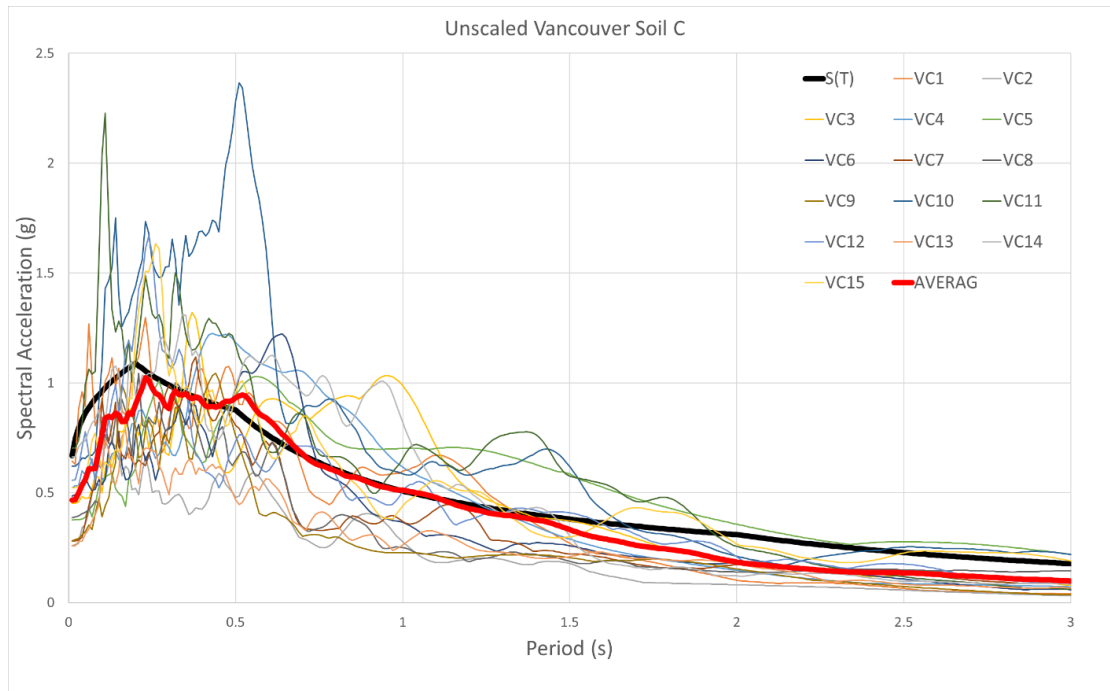


Figure 4-27 Compare of Unscaled ground motions with Vancouver hazard spectrum Class C site

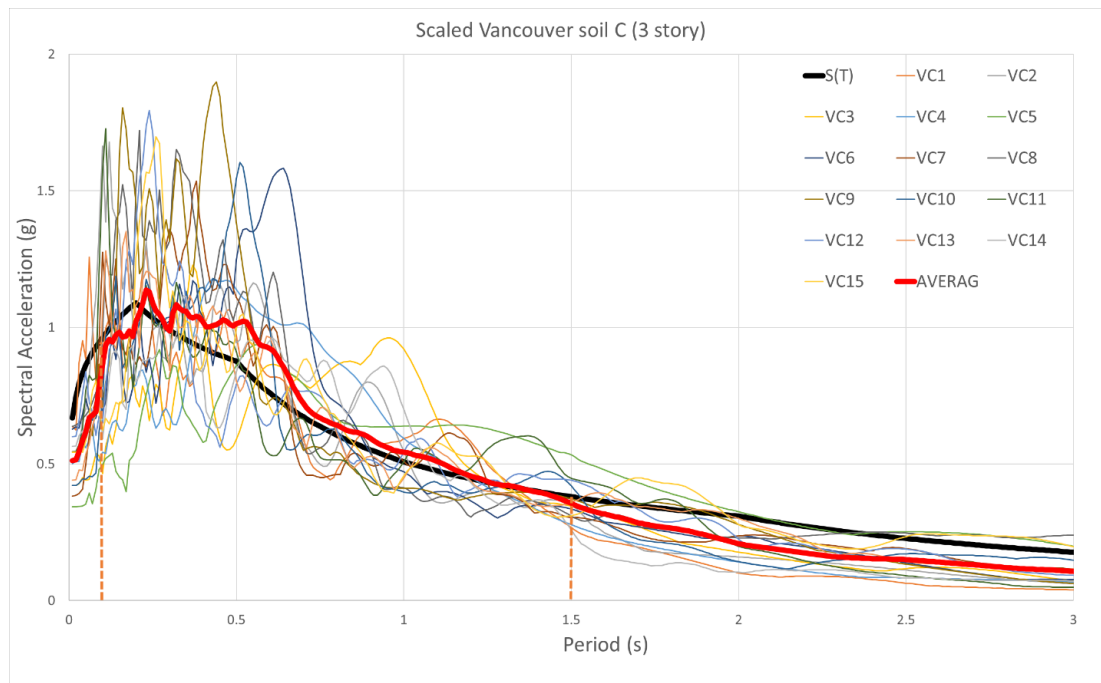


Figure 4-28 Scaled accelerograms for 3-storey building located in Vancouver Class C site

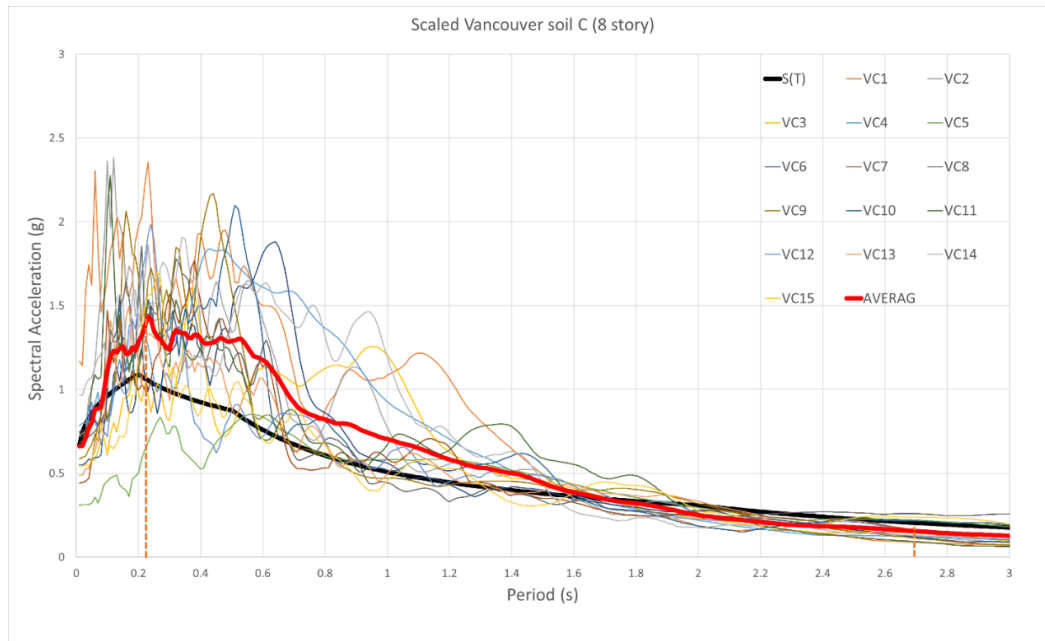


Figure 4-29 First Scaled accelerograms for 8-storey building located in Vancouver Class C site

According to Atkinson and Tremblay (2015), the mean response spectrum should not deviate more than 10% below the target spectrum $S_T(T)$, Therefore second scaled factor equal to 1.15 was applied to all the accelerograms.

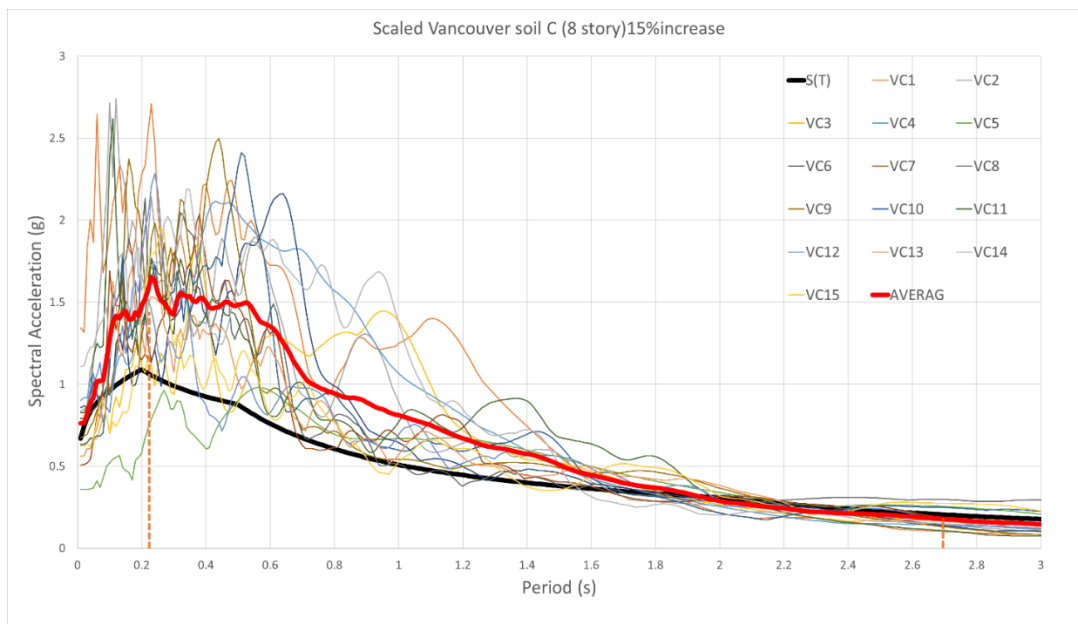


Figure 4-30 Second calibration to reduce the deviation of mean spectrum less than 10% with target spectrum for Vancouver 8-storey Class C site

Vancouver 3 and 8 storey site class E

The scaling procedure was carried out individually for 3 and 8-storey structures owing to their different time spans. While the initial scaling for the Vancouver 3-storey building with soil type E met all the criteria specified in the NBC 2015 user's guide Figure 4-32, the Vancouver 8-storey building with the same soil type exhibited a mean response spectrum deviation exceeding 10% below ST(T) Figure 4-33. As a result, a secondary scaling factor of 1.20 was applied to all the accelerograms to address this inconsistency Figure 4-34.

Table 4-14 Scale factor for Vancouver site class E, 3-and 8-storey buildings.

No	Record ID	Scale factor (3 storey)	Scale factor (8 storey)	Record	EVENT TYPE
1	30	2.03	2.79	RSN728_SUPER.B_B-WSM180.AT2	Crustal
2	205	2.40	2.91	RSN6890_DARFIELD_CMHSN10E.AT2	Crustal
3	207	1.99	2.25	RSN6959_DARFIELD_REHSN02E.AT2	Crustal
4	36	2.60	3.51	RSN758_LOMAP_EMY350.AT2	Crustal
5	37	2.22	3.13	RSN759_LOMAP_A01000.AT2	Crustal
6	208	2.13	3.10	RSN6959_DARFIELD_REHSS88E.AT2	Crustal
7	RBT2	1.16	1.61	2147b_a_Scaled.acc	in-slab
8	RBT1	1.49	1.75	1412c_a_Scaled.acc	in-slab
9	JP71	0.55	0.82	MYG0041103111446.EW	Interface
10	JP243	1.21	1.46	KKWH080309260450.NS1	Interface
11	JP240	0.71	1.10	HKD1260309260450.NS	Interface
12	JP244	1.33	1.49	KKWH080309260450.NS2	Interface
13	JP225	0.93	1.04	HKD1080309260450.EW	Interface
14	JP210	1.22	1.68	HKD1000309260450.NS	Interface
15	JP239	1.27	1.81	HKD1260309260450.EW	Interface

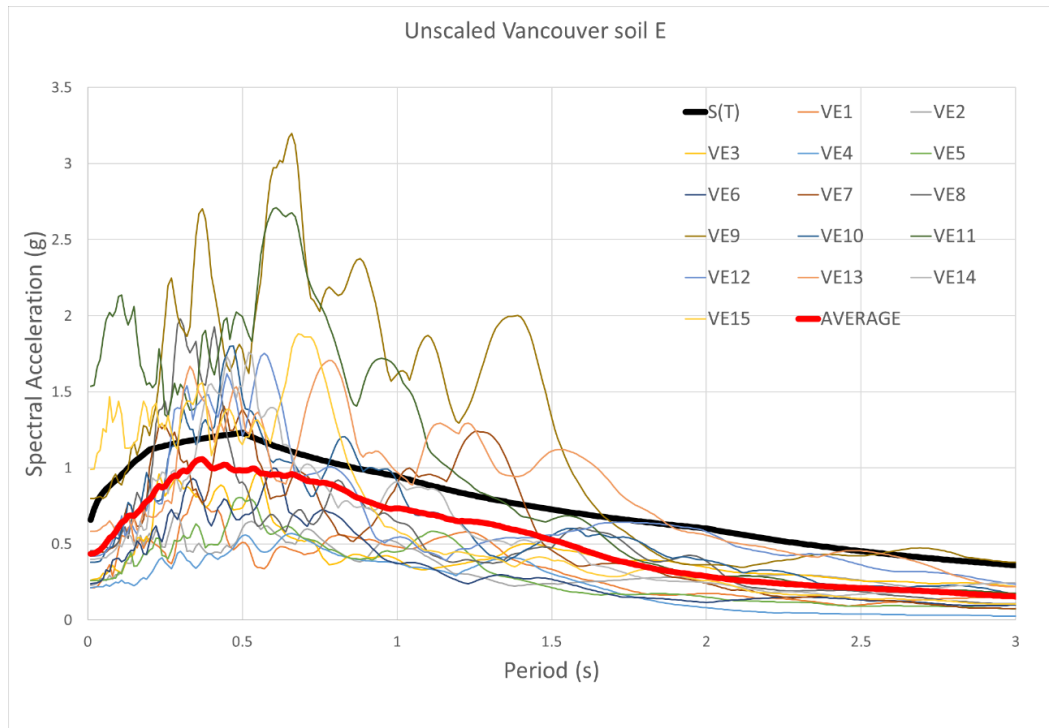


Figure 4-31 Compare of Unscaled ground motions with Vancouver hazard spectrum Class E site

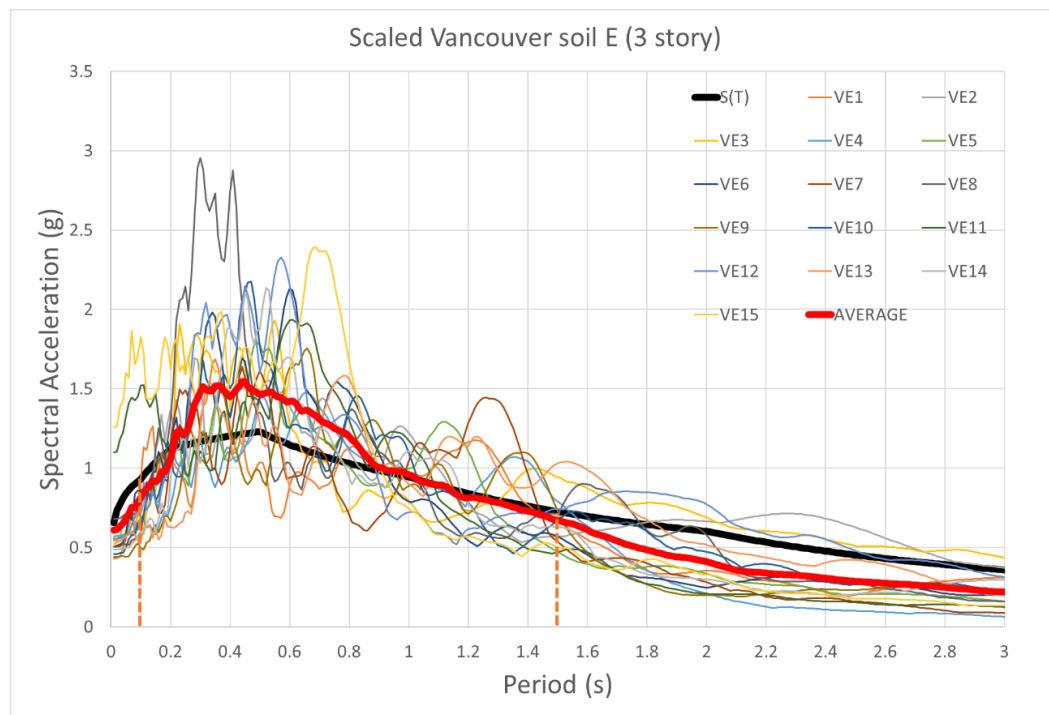


Figure 4-32 Scaled accelerograms for 3-storey building located in Vancouver Class E site

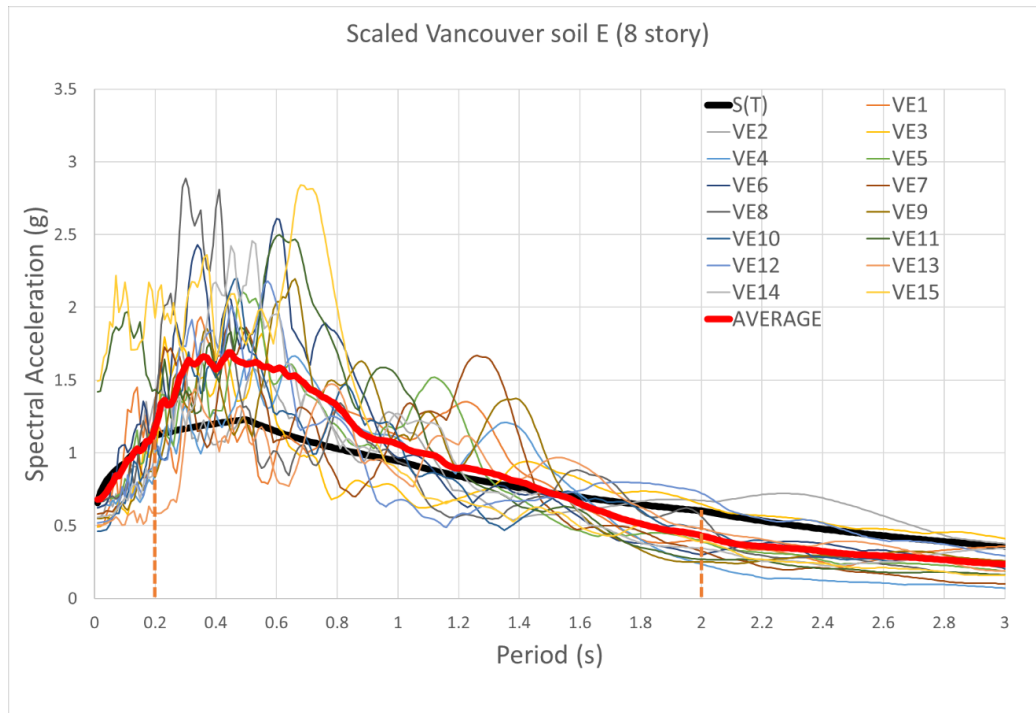


Figure 4-33 First Scaled accelerograms for 8-storey building located in Vancouver Class E site

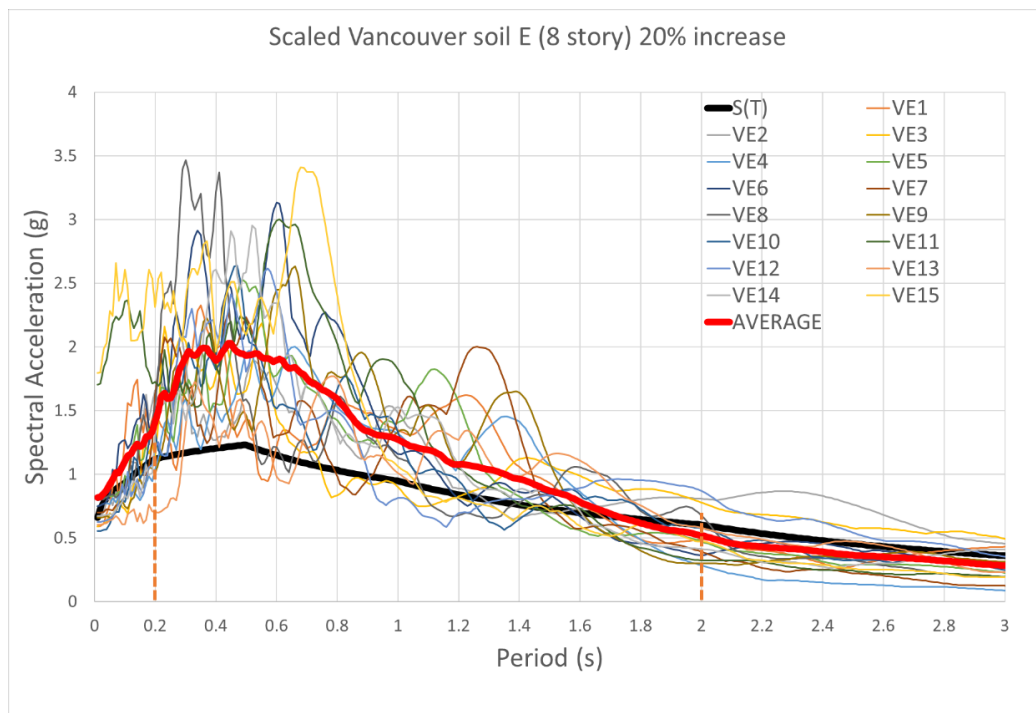


Figure 4-34 Second calibration to reduce the deviation of mean spectrum less than 10% with target spectrum for Vancouver 8-storey Class E site.

4.4.4.2 Second method for Scaling the ground motions

To accommodate a broad period range, calibration factors should be computed based on the specific event type and period range of interest. This approach helps prevent unnecessary demand on the structure. According to the research which were done by Goda and Atkinson (2011), Daneshvar et al. (2015a) The seismic hazard model for western Canada can be specified by the average magnitude and distance measurements for the risk factors associated with each event type. The corresponding values for four periods are presented in Table 4-15 according to the findings of Daneshvar et al. (2015a). As demonstrated, the primary magnitude-distance scenarios for each fault mechanism exhibit minimal variation across the different period range. Using these findings as a basis, a magnitude-distance scenario was established for each event type, along with specific period ranges for each scenario: 0.2-0.8 s for crustal events, 0.3-1.5 s for in-slab events, and 1.0-3.0 s for interface earthquakes (Atkinson and Tremblay, 2015).

Table 4-15 magnitude-distance scenario for each event type.

	T = 0.2 s		T = 0.5 s		T = 1.0 s		T = 2.0 s	
Event Type	M	R (km)	M	R (km)	M	R (km)	M	R (km)
Crustal	6.5	14	<u>6.7</u>	<u>14</u>	6.8	18	7.0	15
In-slab	6.9	61	7.0	56	<u>7.0</u>	<u>52</u>	7.1	51
Interface	8.6	142	8.7	142	8.6	141	<u>8.6</u>	<u>141</u>

The period range was divided in to two scenarios for an 8-storey building situated on the soil types C. the scenario 1 was defined for Crustal and In-slab earthquake type and the scenario 2 was defined for Interface seism type with period range indicated in

Table 4-16.

Table 4-16 Dividing the period range according to earthquake type site class C.

Vancouver 8 story site soil C	Earthquake type	Period range (s)
scenario 1	Crustal - Inslab	0.27 - 1.5
scenario 2	Interface	1.0 - 2.66

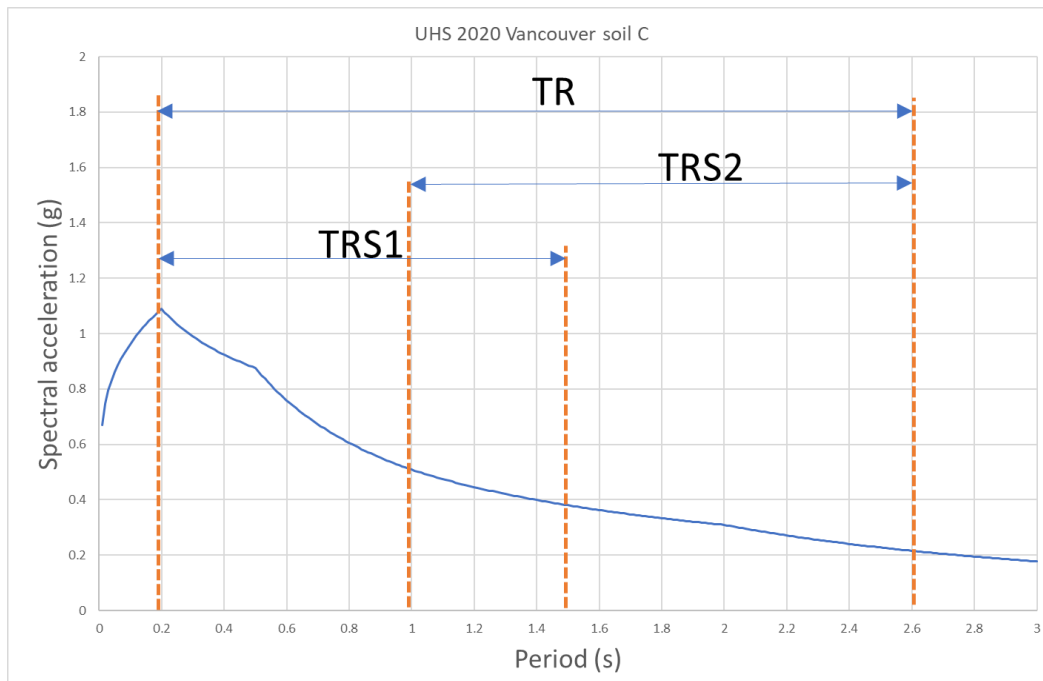


Figure 4-35 Period range of interest for 3-and-8 storey buildings site class C.

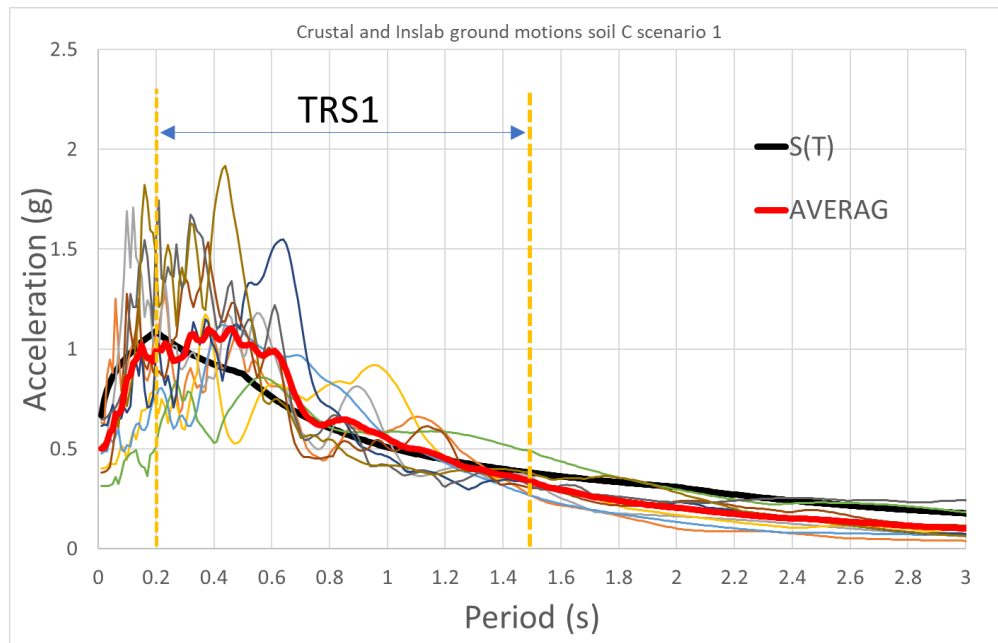


Figure 4-36 Scaling the ground motions within the TRS1 range for site class C.

For the Vancouver 8 storey soil C scenario 2, the mean response spectrum deviates more than 10% below ST(T), There for second scaled factor equal to 1.1 was applied to the Interface ground motions.

Table 4-17 Second scale factor for Vancouver 8-storey Class C site.

No	Record ID	Scale factor 3-storey	Second Scale factor 8-stoeey	Record	EVENT TYPE
1	378	1.97	2.00	RSN801_LOMAP_SJTE315.AT2	Crustal
2	343	0.99	0.99	RSN265_VICT_CPE045.AT2	Crustal
3	425	0.96	0.92	RSN2627_CHICHI.03_TCU076E.AT 2	Crustal
4	488	0.93	0.89	RSN4850_CHUETSU_65013EW.AT 2	Crustal
5	439	0.91	0.83	RSN3748_CAPEMEND_FFS270.AT 2	Crustal
6	374	1.29	1.27	RSN755_LOMAP_CYC285.AT2	Crustal
7	RBT1	1.37	1.37	103l56ol_y0a_Scaled.acc	in-slab
8	RBT2	1.65	1.67	103l56ol_y0b_Scaled.acc	in-slab
9	RBT3	1.82	1.84	0725a_a_Scaled.acc	in-slab
10	JP83	0.68	1.10	MYG0111103111446.EW	Interface
11	JP9	0.77	1.21	FKS0071103111446.EW	Interface
12	JP264	1.71	2.19	KSRH050309260450.NS2	Interface
13	JP231	1.07	1.36	HKD1110309260450.EW	Interface
14	JP232	1.04	1.17	HKD1110309260450.NS	Interface
15	JP227	0.85	1.96	HKD1090309260450.EW	Interface

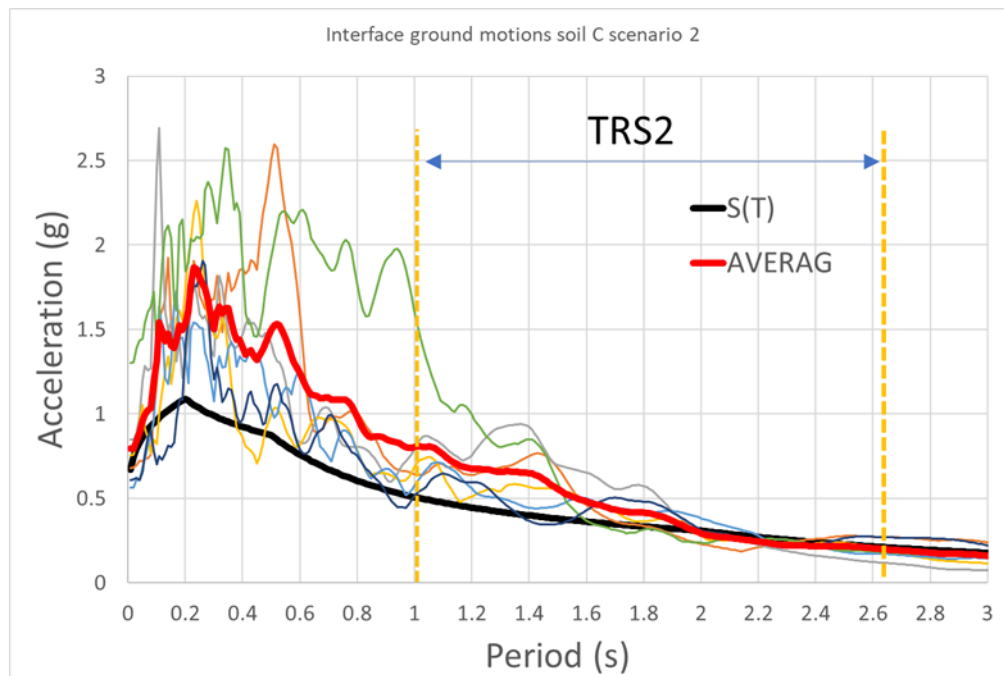


Figure 4-37 Scaling the ground motions within the TRS2 range for site class C.

The period range was segmented into two scenarios for an 8-storey building located on soil types E. Scenario 1 was designated for Crustal and in-slab earthquake types, while Scenario 2 was specified for Interface seismic type, outlining the period range as indicated in Table 4-18.

Table 4-18 Dividing the period range according to earthquake for site class E.

Vancouver 8 story site soil E	Earthquake type	Period range (s)
scenario 1	Crustal - Inslab	0.2 - 1.5
scenario 2	Interface	1.0 - 1.96

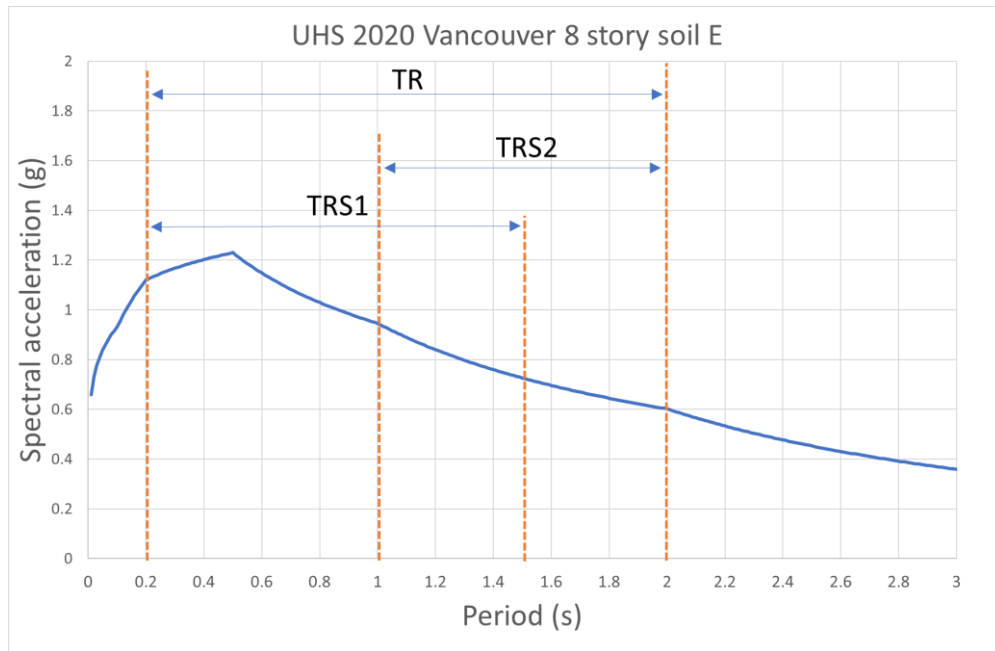


Figure 4-38 Period range of interest for 3-and 8-storey buildings site class E.

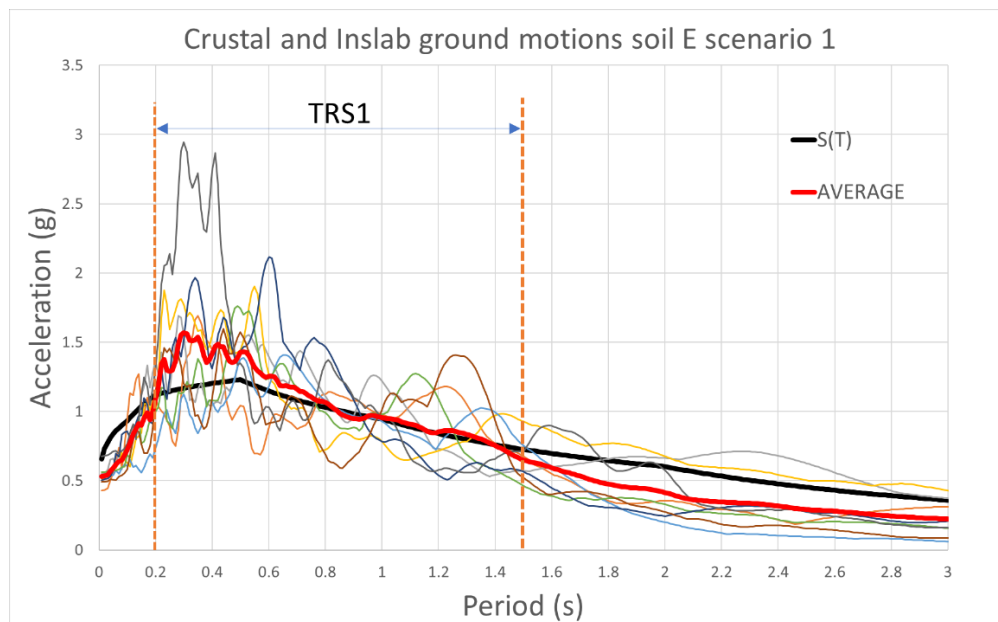


Figure 4-39 Scaling the ground motions within the TRS1 range for site class E.

For the Vancouver 8 storey soil E scenario 2, the mean response spectrum deviates more than 10% below $S_T(T)$, There for second scaled factor equal to 1.1 was applied to the Interface ground motions.

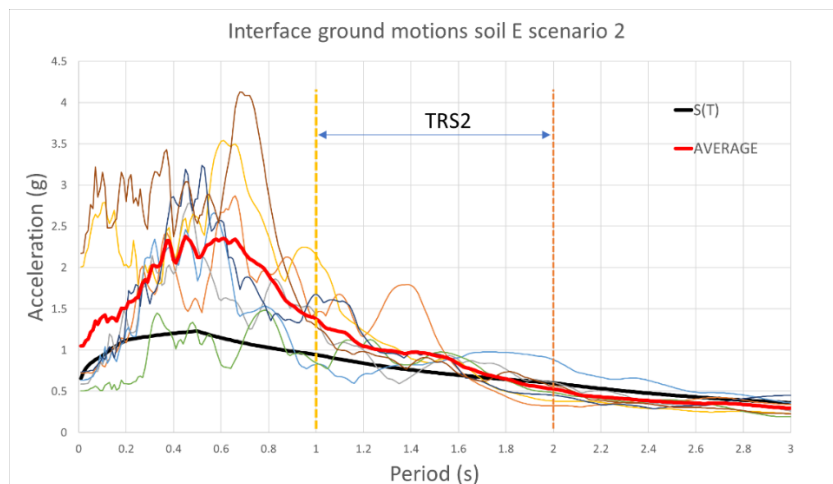


Figure 4-40 Scaling the ground motions within the TRS2 range for site class E.

Table 4-19 Second scale factor for Vancouver 8-storey site class E.

No	Record ID	Scale factor 3-storey	Second Scale factor 8-storey	Record	EVENT TYPE
1	30	2.03	2.03	RSN728_SUPER.B_B-WSM180.AT2	Crustal
2	205	2.40	2.39	RSN6890_DARFIELD_CMHSN10E.AT2	Crustal
3	207	1.99	1.96	RSN6959_DARFIELD_REHSN02E.AT2	Crustal
4	36	2.60	2.48	RSN758_LOMAP_EMY350.AT2	Crustal
5	37	2.22	2.18	RSN759_LOMAP_A01000.AT2	Crustal
6	208	2.13	2.11	RSN6959_DARFIELD_REHSS88E.AT2	Crustal
7	RBT2	1.16	1.14	2147b_a_Scaled.acc	in-slab
8	RBT1	1.49	1.49	1412c_a_Scaled.acc	in-slab
9	JP71	0.55	0.90	MYG0041103111446.EW	Interface
10	JP243	1.21	1.54	KKWH080309260450.NS1	Interface
11	JP240	0.71	1.31	HKD1260309260450.NS	Interface
12	JP244	1.33	1.52	KKWH080309260450.NS2	Interface
13	JP225	0.93	0.87	HKD1080309260450.EW	Interface
14	JP210	1.22	1.84	HKD1000309260450.NS	Interface
15	JP239	1.27	2.19	HKD1260309260450.EW	Interface

CHAPTER 5 RESULTS AND DISCUSSION

In this Chapter, the results obtained from the nonlinear time history analysis are presented and discussed. Two buildings, one with 8-storey and another with 3-storey, located in Vancouver and Montreal considering soil types C and E, were analyzed using the set of accelerograms calibrated to match the Vancouver and Montreal design spectra. The primary objective is to evaluate the impact of soil-structure interaction on the demands placed on both the foundation and superstructure. The response of superstructure-foundation-soil system was tracked through inter-storey drifts, link rotations, link shear force demand, overturning moments on foundations, foundation vertical displacements, and forces in the springs of the Winkler foundation model.

5.1 Results for Vancouver

5.1.1 General comments

To analyze the models, 15 calibrated accelerograms were employed for both a 3-storey and an 8-storey building situated in Vancouver for two site categories (site class C and E). The analysis was carried out for frames with capacity-protected foundation (CP), and a foundation designed without any capacity consideration. Fixed-base case was also considered for comparison. A total of 180 analyses were conducted. For each studied case, the average of the five highest responses was chosen among the results derived from the analysis using the 15 sets of accelerograms and compared against design predictions.

5.1.2 Periods of vibration

The interaction between the soil and structure significantly affects the dynamic response of the system. When a structure is placed on softer soil with a smaller foundation, it experiences greater flexibility and deformation. This increased flexibility results in longer periods of vibration than the structure on a rigid foundation. Conversely, when the foundation is larger and the soil is more rigid, the structure tends to have less flexibility and deformation. Consequently, this configuration typically leads to shorter periods of vibration compared to structures on softer soils with smaller foundations. It's important to note that the characteristics of the soil, such as its stiffness, density, and damping properties, play a crucial role in determining the structural response.

Table 5-1 Period change according to different site class and foundations.

Structure	Site Class	Foundation type	$T_{1,SSI}$ (s)	$T_{1,FIX}$ (s)	$T_{1,SSI}/T_{1,FIX}$
Vancouver 3 storey	C	CP	0.52	0.51	1.03
		NCF	0.53		1.05
	E	CP	0.49	0.44	1.11
		NCF	0.55		1.25
Vancouver 8 storey	C	CP	1.32	1.29	1.02
		NCF	1.34		1.04
	E	CP	1.24	0.99	1.25
		NCF	1.28		1.29

In all scenarios, the structures experienced longer periods when considering soil-structure interaction. The highest increase was observed for the Vancouver 8-storey structure on site Class E, with an approximate 29% increase, while the smallest increase was noted for the Vancouver 3-storey structure on site Class C, with about a 3% increase. The interaction between the soil and structure significantly affects the dynamic response of the system. When a structure is placed on softer soil with a smaller foundation, it experiences greater flexibility and deformation. This increased flexibility results in longer periods of vibration than the structure on a rigid foundation. Conversely, when the foundation is larger and the soil is more rigid, the structure tends to have less flexibility and deformation. Consequently, this configuration typically leads to shorter periods of vibration compared to structures on softer soils with smaller foundations. It's important to note that the characteristics of the soil, such as its stiffness, density, and damping properties, play a crucial role in determining the structural response.

Table 5-1, shows that soil-structure interaction has a more significant influence on structural period when the foundation is located on site class E compared to site class C. For a 3-storey structure, an increase ranging between 3% to 5% is recorded for site Class C while this difference ranges between 11% to 25% for site class E. Similar trends is observed for 8-storey structures. The size of the foundation also impacts the period; by decreasing the size from capacity-protected to non-capacity-factored, the period is reduced by up to 2% to 3%.

5.1.3 Lateral displacement

According to NBCC 2020 the largest inter-storey deflection at any level shall be limited to $0.01h_s$ for post-disaster buildings, $0.02h_s$ for high importance category buildings, and $0.025h_s$ for all other buildings, where h_s represents the height of a storey. In this project, the limit of $0.025h_s$ is considered. Understanding the inter-storey drift in relation to soil-structure interaction is crucial in assessing the building's stability and performance. Foundation rotation may cause significant additional drift along the building's height, with its magnitude dependent heavily on both soil characteristics and the size of the foundation. As soil rigidity and foundation size increase, there is a notable reduction in foundation rotations, thereby affecting less the overall inter-storey drift of the superstructure. Therefore, balancing soil conditions and foundation dimensions becomes instrumental in mitigating excessive inter-storey drift and ensuring structural stability.

Table 5-2 Inter-storey drift percentage for 3-storey structure Class C and E sites.

Structure	Site Class	Foundation type	Storey		
			1	2	3
Vancouver 3 storey	C	Fixed base	0.95%	0.74%	0.37%
		CP	0.89%	0.86%	0.41%
		NCF	0.99%	0.95%	0.56%
	E	Fixed base	1.05%	0.91%	0.36%
		CP	0.99%	0.98%	0.46%
		NCF	1.06%	1.06%	0.58%

For 3-storey buildings located in Vancouver on sites Class C and E, the frame with the foundation designed without any capacity protection considerations, experienced the largest inter-storey drifts. Frames with CP foundation developed smaller inter-storey drifts and the foundation rotates less and the additional drift along the height of the building is smaller. The maximum inter-storey drift occurred in first level for both site Classes reaching 0.99% and 1.06% for site Class C and E, respectively. In all cases, the inter-storey drifts were smaller than the design limit.

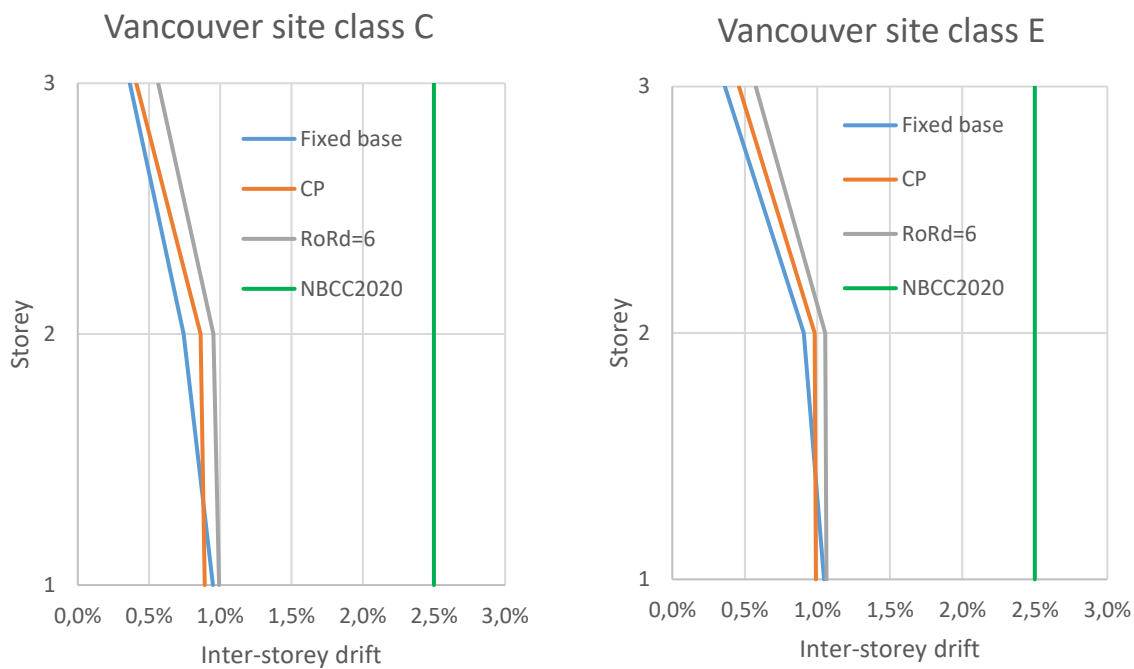


Figure 5-1 Inter-storey drift for Vancouver 3-storey site class C and E

For 8-storey buildings situated in Vancouver on site class C and E, foundations designed without a capacity protection exhibited the most pronounced inter-storey drifts, but they still remained below the 2.5% h threshold specified in the NBCC 2020 requirements. These foundations were smaller in size compared to CP foundations, resulting in increased rotation and additional drift along the building's height. Figure 5-2 illustrates that the inter-storey drift increases as the size of the foundation decreases. The 8-storey building on site Class C exhibits the largest inter-storey drift at the 5th level, reaching approximately 1.6%, while the maximum drift index of 2% is recorded at the 6th level for class E site.

Table 5-3 Inter-storey drift percentage for 8-storey structure site class C and E

Structure	Site Class	Foundation type	Storey							
			1	2	3	4	5	6	7	8
Vancouver 8 storey	C	Fixed base	0.96%	0.98%	1.02%	1.13%	1.32%	1.17%	1.43%	0.77%
		CP	0.69%	1.08%	1.15%	1.25%	1.43%	1.23%	1.51%	0.82%
		NCF	0.77%	1.13%	1.17%	1.36%	1.58%	1.30%	1.52%	0.89%
	E	Fixed base	1.20%	1.11%	1.18%	1.29%	1.39%	1.62%	1.59%	0.88%
		CP	0.97%	1.36%	1.54%	1.65%	1.66%	1.76%	1.76%	1.15%
		NCF	1.17%	1.59%	1.79%	1.90%	1.90%	2.00%	1.93%	1.39%

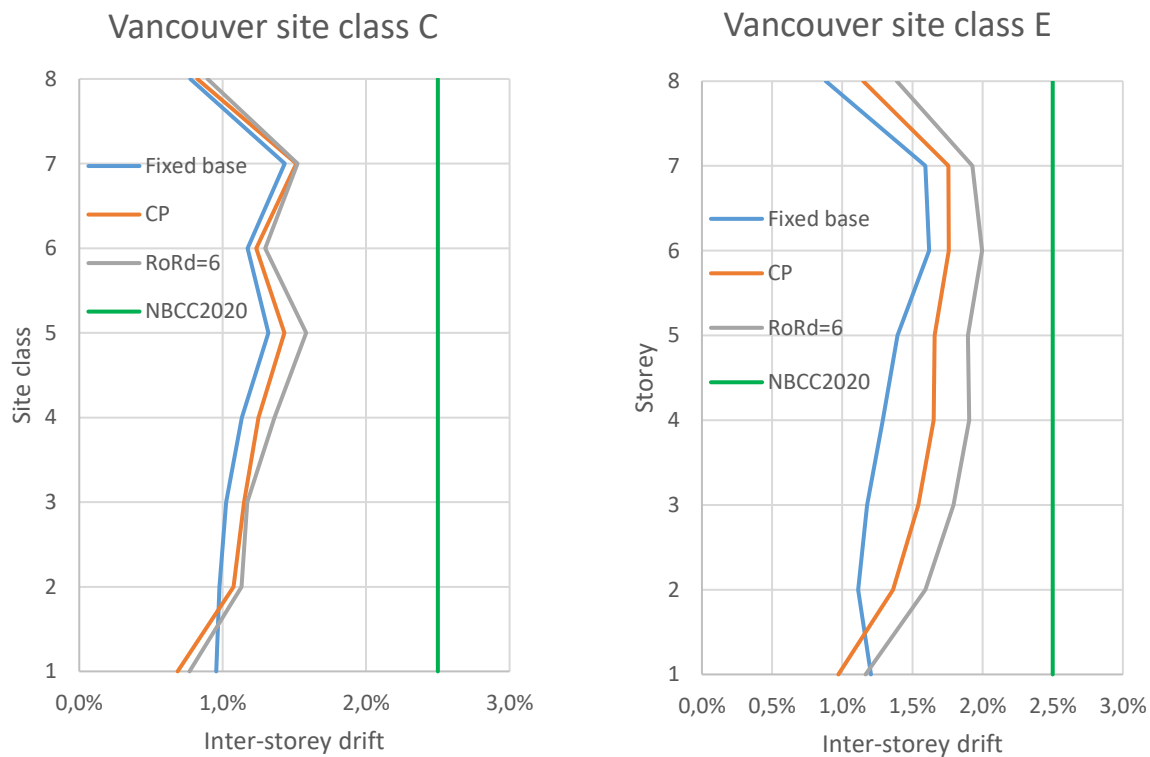


Figure 5-2 Inter-storey drift for Vancouver 8-storey site class C and E

5.1.4 Overturning moment

The magnitude of overturning moments is influenced by several factors, including the seismic frequency content, structural mass, stiffness, foundation dimensions, and SSI effects. Each of these aspects has the potential to either increase or decrease this response parameter. NBCC2020 and CSA A23.3-19 provide methods to predict the magnitude of overturning moments, detailed in section 2.2.1. The M_d design, which represents the moment resulting from all yielded and strain-hardened links, was utilised in the design of the CP foundation. An $M_{RdRo=6}$ was employed to design foundation without considering any capacity design (NCF). Corresponding footing dimensions were implemented in the numerical model. Overturning moment, M_{SSI} was obtained from nonlinear time history analysis. Result of the analysis shows that the most significant yielding of the links was observed for the fixed-base scenario. This overturning moment is referred to in the text as the actual probable moment M_{FB} whose magnitude is situated between the values of M_d and $M_{RdRo=6}$.

Table 5-4 Overturning moment for different type of foundations

Structure	Site Class	Foundation type	Design	Fixed base	Full model	M_d/M_{FB}	M_{SSI}/M_{FB}
			M_d (kN.m)	M_{FB} (kN.m)	M_{SSI} (kN.m)		
Vancouver 3 storey	C	CP	16719	18472	18338	0.91	0.99
		NCF	7672	18472	16055	0.42	0.87
	E	CP	23448	28201	27685	0.83	0.98
		NCF	11189	28201	29110	0.40	1.03
Vancouver 8 storey	C	CP	66735	71318	69549	0.94	0.98
		NCF	30108	71318	65634	0.42	0.92
	E	CP	126472	151432	145148	0.84	0.96
		NCF	67273	151432	136023	0.44	0.90

As to evaluate the influence of foundation sizing on overturning moment and energy dissipation, a different scenario was considered to see how soil-structure interaction decreases the overturning

moment. It can be observed in Table 5-4 that the ratio M_{fSSI}/M_{fFB} reduces between 5% to 12% when the size of the foundations is reduced. It can also be seen that for the 8-storey frames on class E site the effect of SSI is more pronounced and $M_{RdRo=6}$ is about 10% smaller than M_{fFB} while this difference is about 8% for class C site frame.

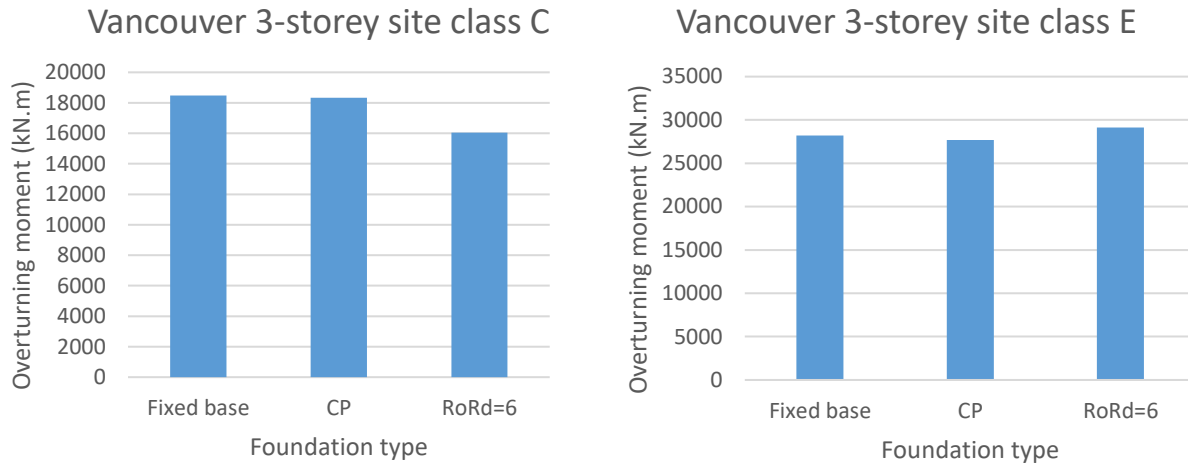


Figure 5-4 Comparison the magnitude of overturning moment for 3-storey site class C and E

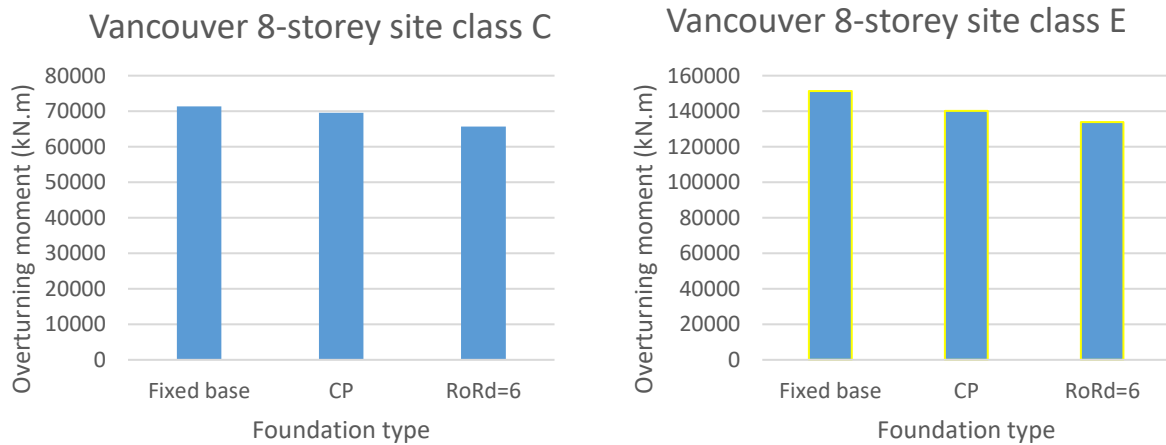


Figure 5-3 Comparison of the magnitude of overturning moment for 8-storey site class C and E

5.1.5 Link inelastic rotation

The shear rotation of the link " γ ," represents the angle between the link and the outer beam. This global rotation, contains an "inelastic (or plastic) part" denoted by " γ_p ." Generally, the inelastic rotational capacity of the link, " γ_{pmax} ," refers to the utmost inelastic rotational angle permitted by ductile response of the links. According to CSA S16-19 the link inelastic rotation γ_{pmax} for shear links should not exceed 0.08 rad. Rocking foundation reduces this rotation due to SSI effect. Therefore, by changing the system from the one having a fixed base to the one having capacity-protected foundation or the foundation with no capacity factor, the inelastic rotation is generally diminished.

Table 5-5 Inelastic rotations for 3-storey building in Vancouver on site class C and E

Structure	Site Class	Foundation type	γ (rad)		
			1	2	3
Vancouver 8 storey	C	Fixed base	0.097	0.062	0.005
		CP	0.090	0.078	0.007
		NCF	0.080	0.071	0.004
	E	Fixed base	0.097	0.073	0.007
		CP	0.082	0.071	0.008
		NCF	0.080	0.072	0.007

Table 5-5 shows that the inelastic rotation reached 0.097 in the first storey, exceeding the requirement of 0.08 stipulated in CSA S16-19. The γ_p is decreased from 0.097 to 0.080 by transitioning from a fixed base to a foundation with no capacity factor.

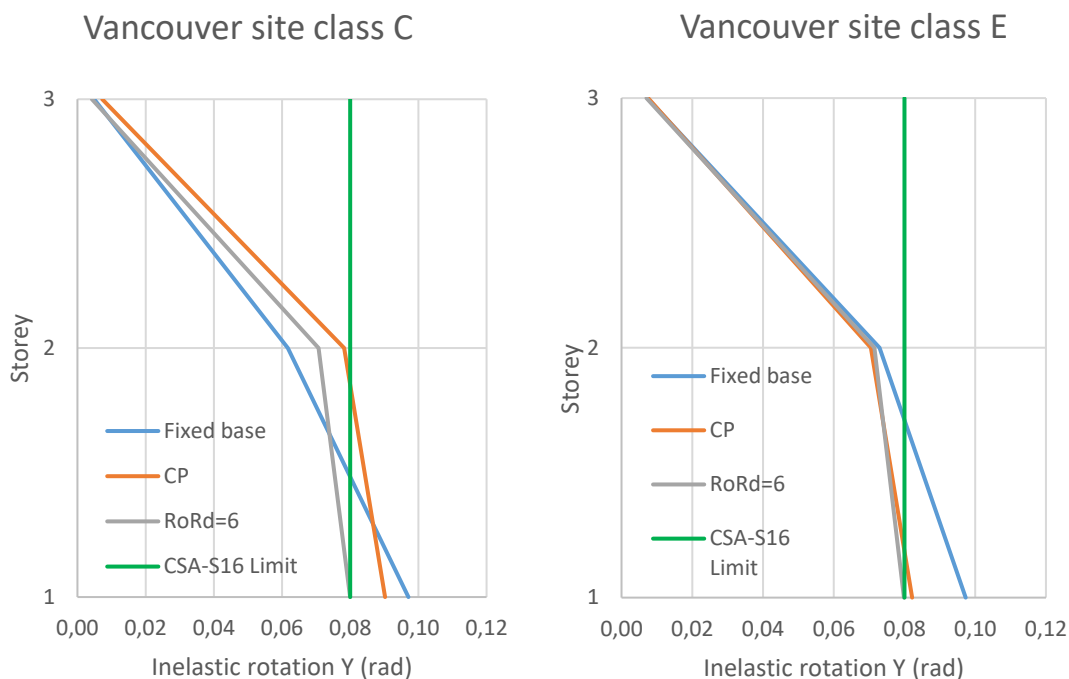


Figure 5-5 Inelastic rotation for 3-storey building in Vancouver on site class C and E

As seen in Table 5-6, for 8-storey building on site Class C, the biggest inelastic rotation of approximately 0.120 rad occurred at the 5th storey with a foundation type of $R_d R_o = 6$, this exceeding the 0.08 rad limit. Conversely, the smallest rotation was observed at the 8th level, ranging between 0.009 and 0.010 rad. Note that in the fixed-base model, the rotation at the first level is higher than in other foundation types. For an 8-storey structure in Vancouver on site Class E, the maximum inelastic rotation occurred in the fixed base model at the 6th level, while the smallest rotation varied between 0.019 and 0.021 was recorded at the 8th level. It's worth noting that for both soil types C and E, the biggest rotations occurred between levels 4 and 7.

Table 5-6 Inelastic rotations for an 8-storey building in Vancouver on site class C and E

Structure	Site Class	Foundation type	γ (rad)							
			1	2	3	4	5	6	7	8
Vancouver 8 storey	C	Fixed base	0.086	0.074	0.090	0.093	0.108	0.078	0.094	0.009
		CP	0.048	0.084	0.087	0.098	0.118	0.081	0.098	0.010
		NCF	0.041	0.073	0.073	0.096	0.120	0.078	0.094	0.009
	E	Fixed base	0.096	0.085	0.081	0.087	0.090	0.104	0.097	0.019
		CP	0.034	0.081	0.072	0.091	0.092	0.096	0.095	0.023
		NCF	0.026	0.074	0.063	0.084	0.088	0.091	0.093	0.021

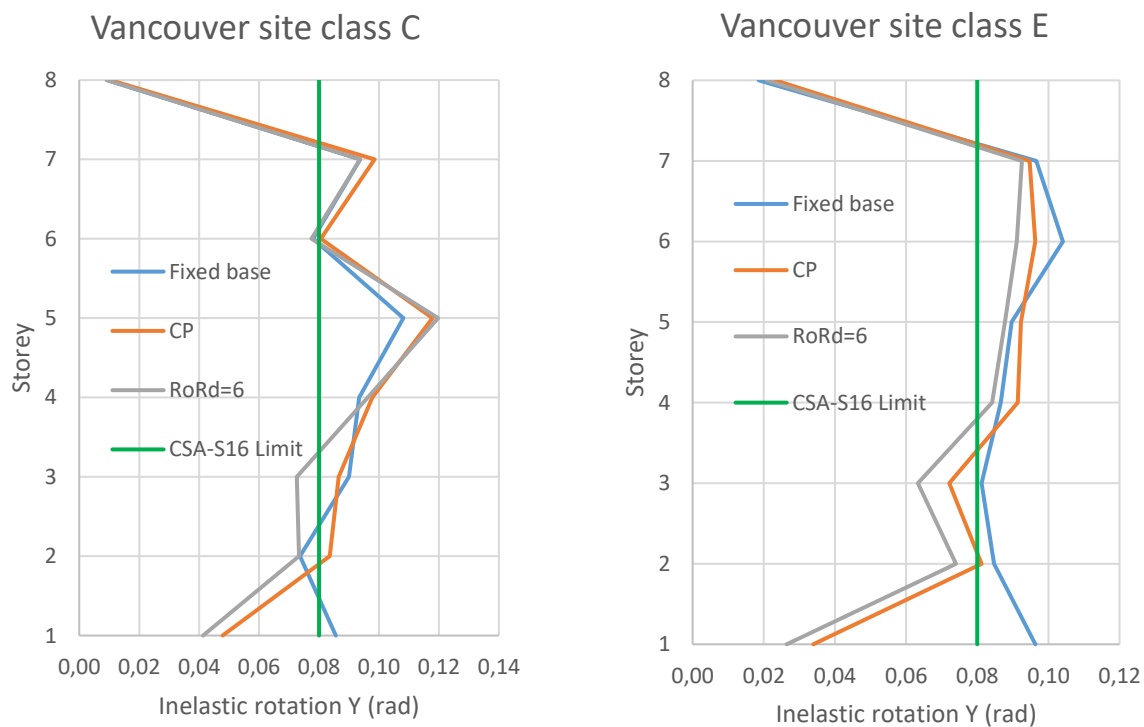


Figure 5-6 Inelastic rotation for 8-storey building in Vancouver on site class C and E

5.1.6 Shear yielding capacity of ductile links

The shear yielding capacity of ductile link without considering the hardening stage of steel 02 was set equal to $V_{pr} = V_p R_y$ where $V_p = 0.55 A_v F_y$, $F_y = 345$ MPa and $R_y = 1.1$ with a goal to estimate the $V_{prob} = \Omega V_{pr}$ and to see how much this V_{prob} will go beyond the V_{pr} . The ratio Ω thus gives the strain hardening potential of the link. According to CSA S16-19 the probable resistance of shear link is equal to $V_{prob} = 1.30 R_y V_p$. Factors 1.3 mainly represent the effect of strain hardening.

For a 3-storey building on site Class C, the highest Ω rate occurs at the first level of the fixed base model, reaching 1.31. This ratio decreases to 1.26 for the NCF($R_d R_o = 6$) foundation due to the consideration of SSI (Soil-Structure Interaction) and the reduction in the foundation size. The lowest value of Ω is equal to 1.03 and occurs for NCF foundation.

In the case of a 3-storey structure situated on Class E site, the biggest Ω value is observed at the first floor level of the fixed-base model, measuring 1.41. This value decreases to 1.34 in the NCF model as a result of SSI (Soil-Structure Interaction) and reducing the foundation's size. The smallest Ω value, amounting to 1.03 in 3th level, is noted within the NCF model.

Table 5-7 The ratio of Ω for 3 storey structure site class C and E

Structure	Site Class	Foundation type	Storey V_{fmax}/V_{pr} (Ω)		
			1	2	3
Vancouver 8 storey	C	Fixed base	1.31	1.19	1.03
		CP	1.30	1.22	1.04
		NCF	1.26	1.20	1.03
	E	Fixed base	1.41	1.33	1.04
		CP	1.35	1.31	1.03
		NCF	1.34	1.33	1.03

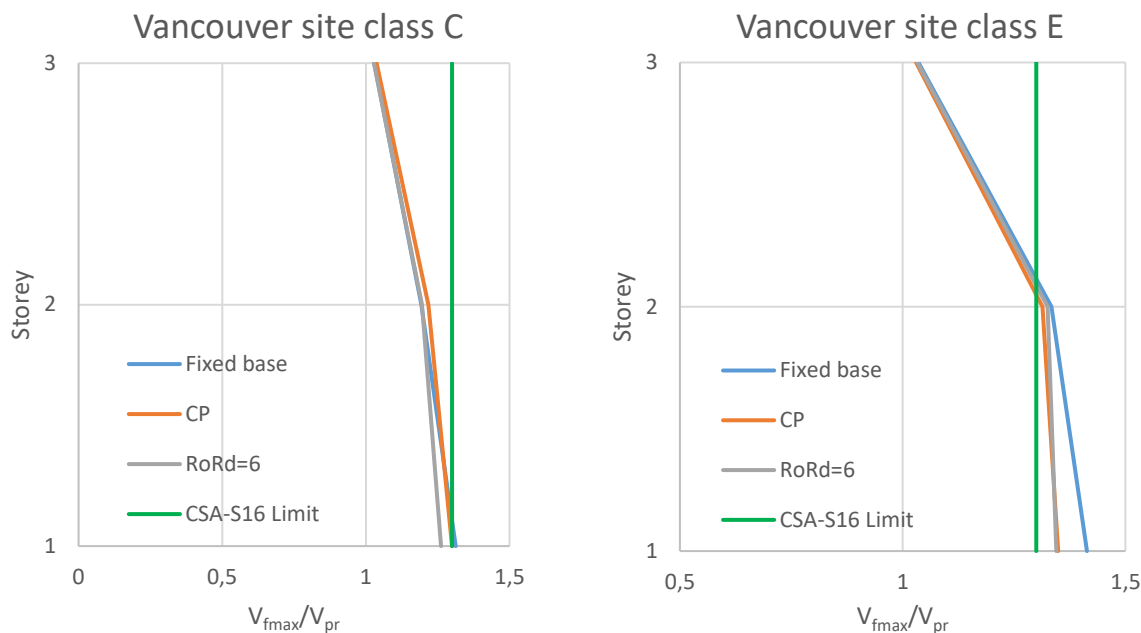


Figure 5-7 The ratio of V_{fmax}/V_{pr} for 3-storey structures on Site Class C and E

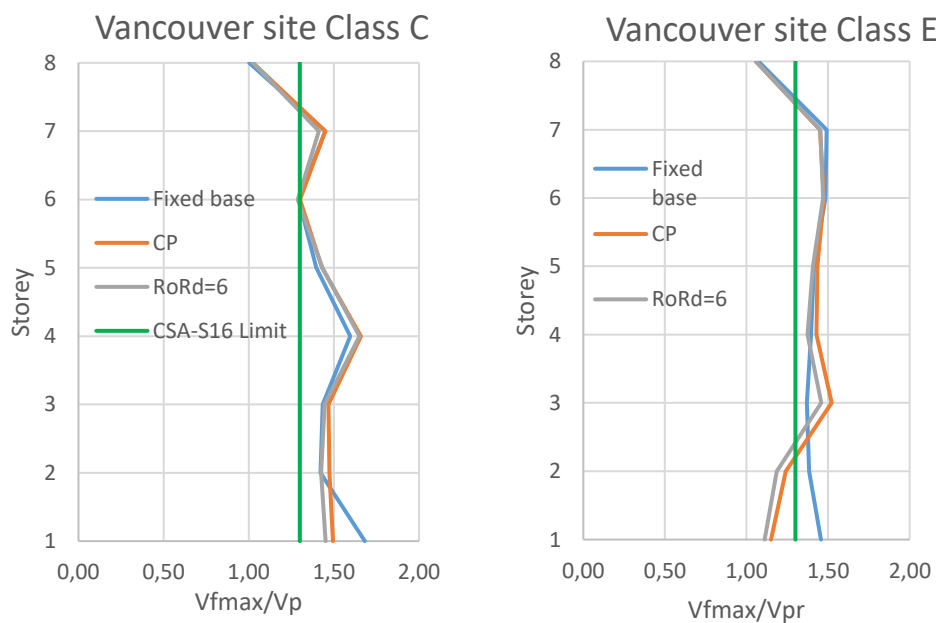
In an 8-storey building situated on Class C site, the maximum Ω value, is observed at the first level within the fixed base model, measuring 1.68. Similar trend is recorded for the 3-storey structure on the same Class site. This value decreases to 1.45 in the NCF model due to effects of SSI and the reduction in foundation dimensions.

For the 3-storey building located on Class E site, the largest Ω is seen on the 7th floor level for the fixed base model, measuring 1.49. This value decreases to 1.45 in the NCF model due to the influence of SSI and the reduction in the foundation's size. The lowest Ω value, equal to 1.06, is observed for the non-capacity factored model.

The impact of SSI is more pronounced for frames on class E site. It can be observed that the Ω factor decreases by a maximum of 31% for this case, whereas the reduction is approximately 16% for the frame on class C site. In many instances, the overstrength factor Ω exceeds the coefficient of 1.3 predicted in CSA S16-19. Hence, it could be argued that the overstrength ratio is somewhat underestimated in the Canadian steel design provisions.

Table 5-8 The ratio of Ω for 8 storey structure soil C and E

Structure	Site Class	Foundation type	Storey $V_{fmax}/V_{pr} (\Omega)$							
			1	2	3	4	5	6	7	8
Vancouver 8 storey	C	Fixed base	1.68	1.42	1.43	1.60	1.40	1.29	1.45	1.00
		CP	1.49	1.47	1.47	1.66	1.43	1.30	1.45	1.02
		NCF	1.45	1.42	1.45	1.65	1.43	1.29	1.41	1.03
	E	Fixed base	1.46	1.38	1.37	1.40	1.41	1.48	1.49	1.07
		CP	1.15	1.24	1.52	1.43	1.44	1.47	1.45	1.06
		NCF	1.11	1.19	1.46	1.37	1.41	1.47	1.45	1.06

Figure 5-8 The ratio of V_{fmax}/V_{pr} for 8-storey structures on Site Class C and E

5.1.7 Soil bearing demands

Load-bearing capacity is the ability of the soil to bear the weight of the structure without excessive settlement or failure. Different soil types exhibit distinct behaviours that significantly influence foundation design. In this project, two different types of soils were considered in the foundation design procedure, corresponding to site Class C and E, defined in NBCC2020.

To assess the soil demand during seismic activity, a numerical model of soil was created by defining springs beneath the foundation. The foundation surface was segmented into multiple springs with varying stiffness. The demand on each of these springs could be measured and compared with the soil bearing capacity q_{ult} to confirm whether the values fall within an acceptable range or not. To assess the results, the software output was normalized by dividing q_{fmax} by q_{ult} . Figure 5-9 illustrates normalized soil bearing along the length of the foundation for Site Classes C and E. These curves depict the maximum normalized compressive force at each spring. For the 3-storey frame site with site Class C, the maximum ratio of q_{max}/q_{ult} is approximately 0.13 for CP foundation and 0.33 for non-capacity factored foundation. On Site Class E, the maximum ratio for CP foundation is 0.6, and for NCF ($R_d R_o=6$) foundation, it is 0.65. Furthermore, the maximum stress occurs at the end of the foundation length, while the minimum stress is situated in the middle.

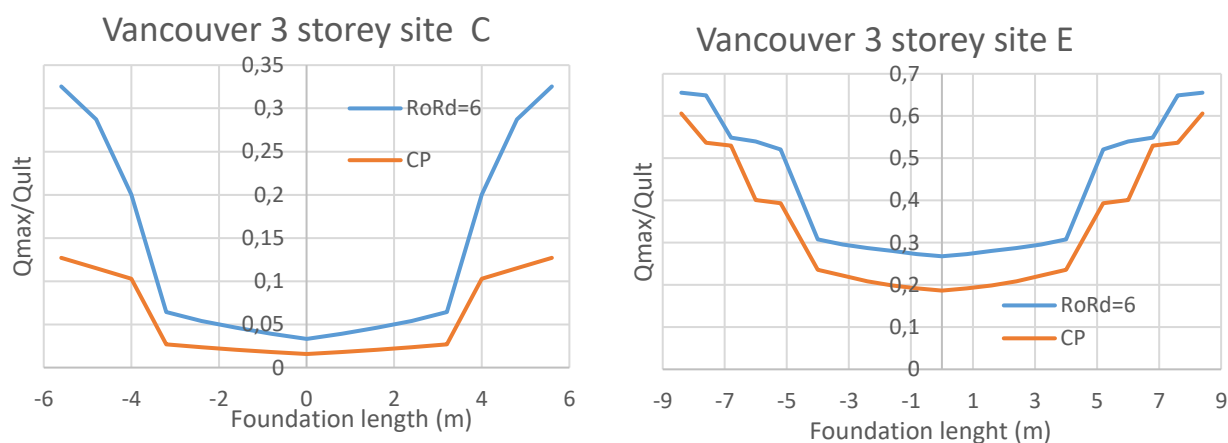


Figure 5-9 Normalized soil stress bearing for 3-storey building on site class C and E

For the 8-storey frame with Class C site, the maximum ratio of q_{max}/q_{ult} is approximately 0.16 for CP and 0.29 for NCF foundation. On Site Class E, the maximum soil stress ratio is 0.52 for CP and

0.65 for NCF foundation. Furthermore, the minimum ratio of soil stress occurs in the middle of the foundation, measuring 0.29 for foundation type CP and 0.35 for NCF foundation.

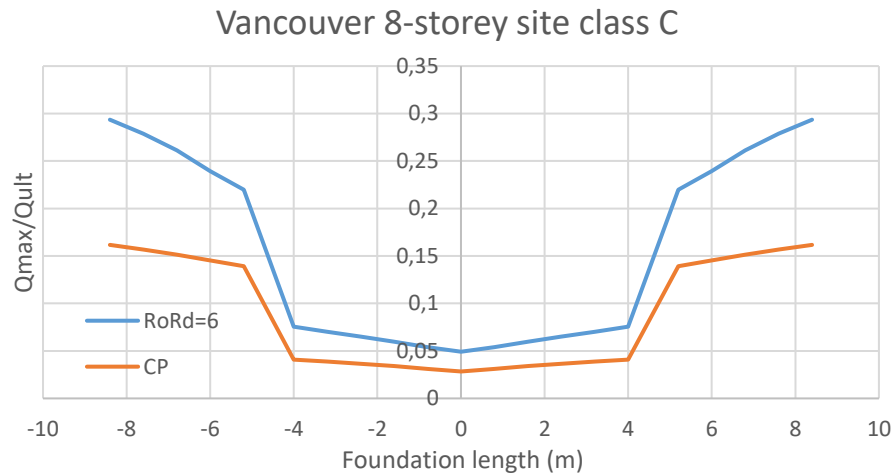


Figure 5-10 Normalized soil stress bearing for 8-storey building on site class C

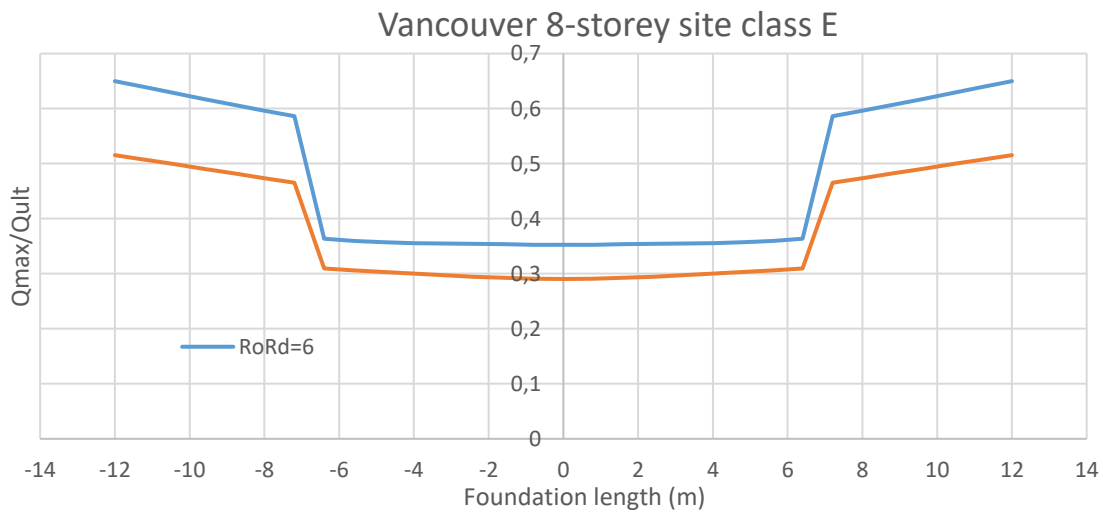


Figure 5-11 Normalized soil stress bearing for 8-storey building on site class E

5.1.8 Settlements and uplift

Settlement and uplift characterize the foundation's rotation and vertical movement, both of which are directly influenced by soil characteristics. This phenomenon tends to be more noticeable on site Class E compared to site Class C. In addition to variable vertical movement, there may also be permanent settlement, which ideally should not exceed 25 mm under any circumstance (Bjurrum 1963).

Monitoring and controlling this permanent settlement is critical to ensure the structural integrity and long-term stability of the foundation and the overall structure. In Figure 5-12, For a 3-storey building class C site Figure 5-12, the highest uplift is observed in the NCF foundation, reaching 21mm, whereas approximately 1.5mm was obtained for the CP foundation. The largest settlement was recorded for the NCF foundation, reaching -6mm, compared about -2mm recorded for the CP foundation. It can be observed that by reducing the size of the foundation both settlement and uplift increased suggesting that in this case SSI have stronger influence and resulting consequently in more significant energy dissipation.

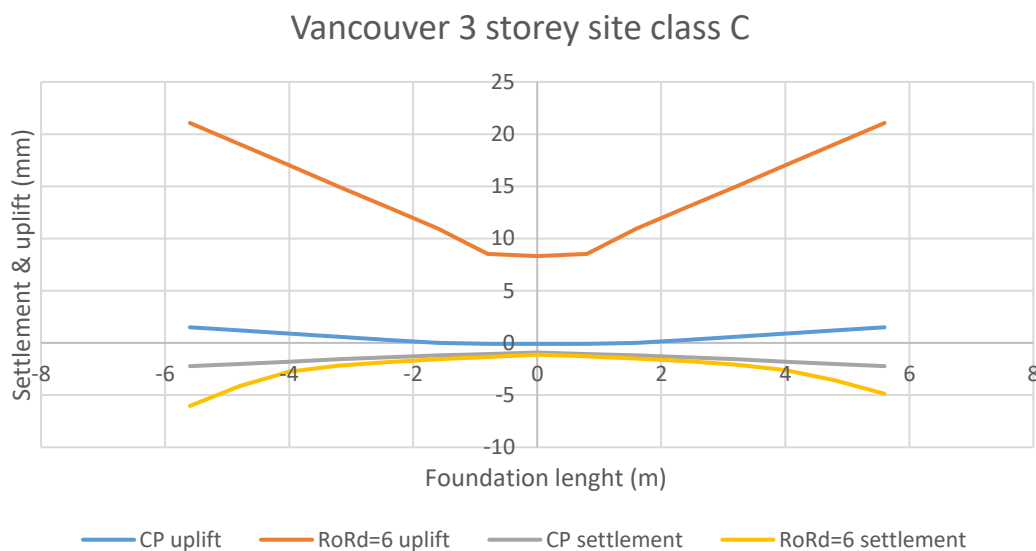


Figure 5-12 Foundation settlement and uplift for a 3-storey building on site class C

As seen in Figure 5-13, for a 3-storey building situated on Class E site, the NCF foundation type displays the highest uplift, reaching 5.9mm, while about 1.8mm was recorded for the CP

foundation. In terms of settlement, the NCF foundation demonstrates the largest settlement, measuring -31.2mm, in contrast to approximately -17.6mm observed for the CP foundation.

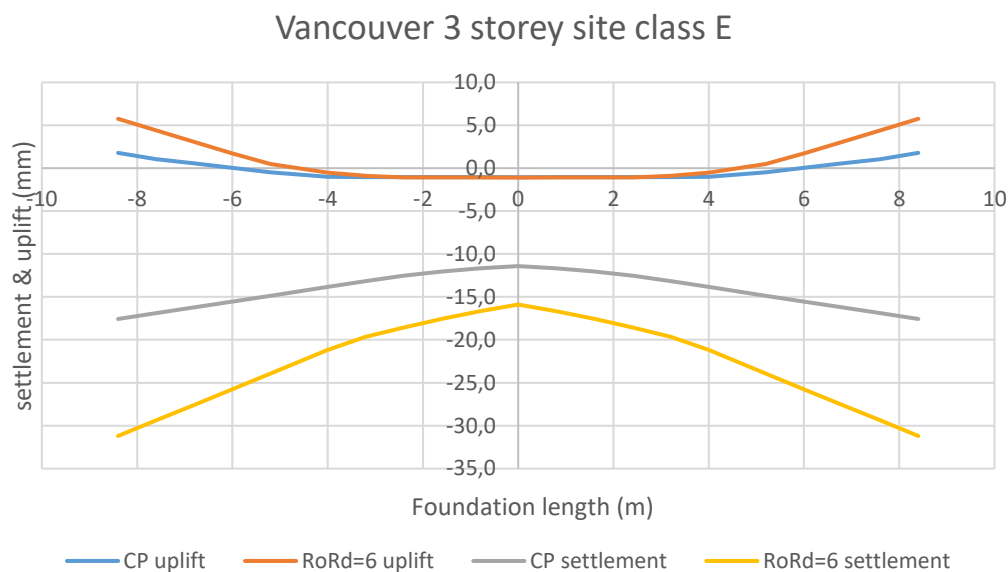


Figure 5-13 Foundation settlement and uplift for a 3-storey building on Soil Class E site

As shown in Figure 5-14, for 8-storey building situated on a Class C site, the NCF ($R_d R_o=6$) foundation demonstrates the highest uplift, reaching 24mm, in contrast to approximately 6mm for the CP foundation. The settlement for the NCF foundation is more significant, reaching -8mm, compared to approximately -4mm observed for the CP foundation. 8-storey building located on a Class E site encountered greater temporary and permanent settlement and uplift. the highest uplift equal to 33mm was observed for the NCF foundation, while approximately 22mm was obtained for the CP foundation. NCF foundation experiences the biggest settlement, reaching -101mm, as opposed to approximately -61mm for the CP foundation. Additionally, the permanent settlement

exceeds Acceptable limits, measuring about -37mm for the CP foundation and -58mm for the $R_dR_o=6$ foundation, surpassing the 25mm threshold, which is deemed unacceptable.

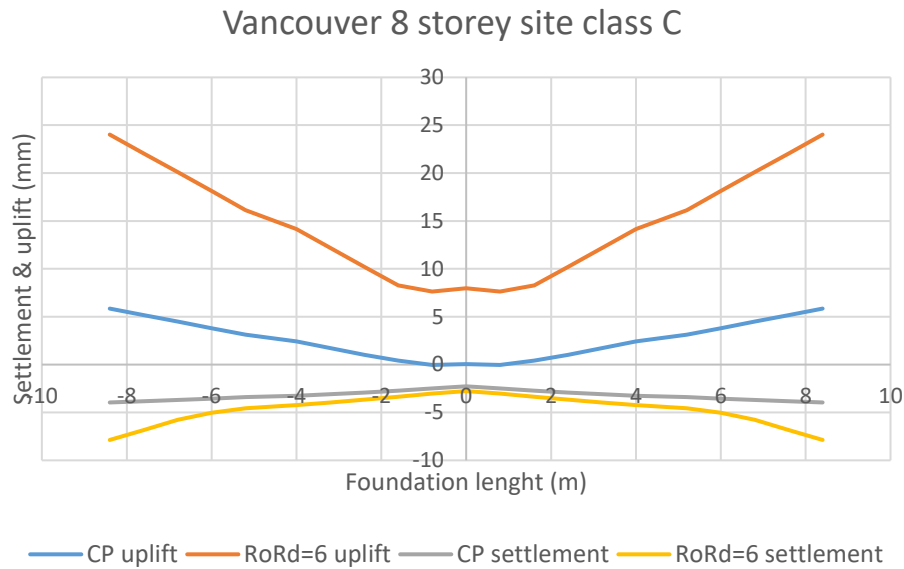


Figure 5-14 Foundation settlement and uplift for an 8-storey building on site class C

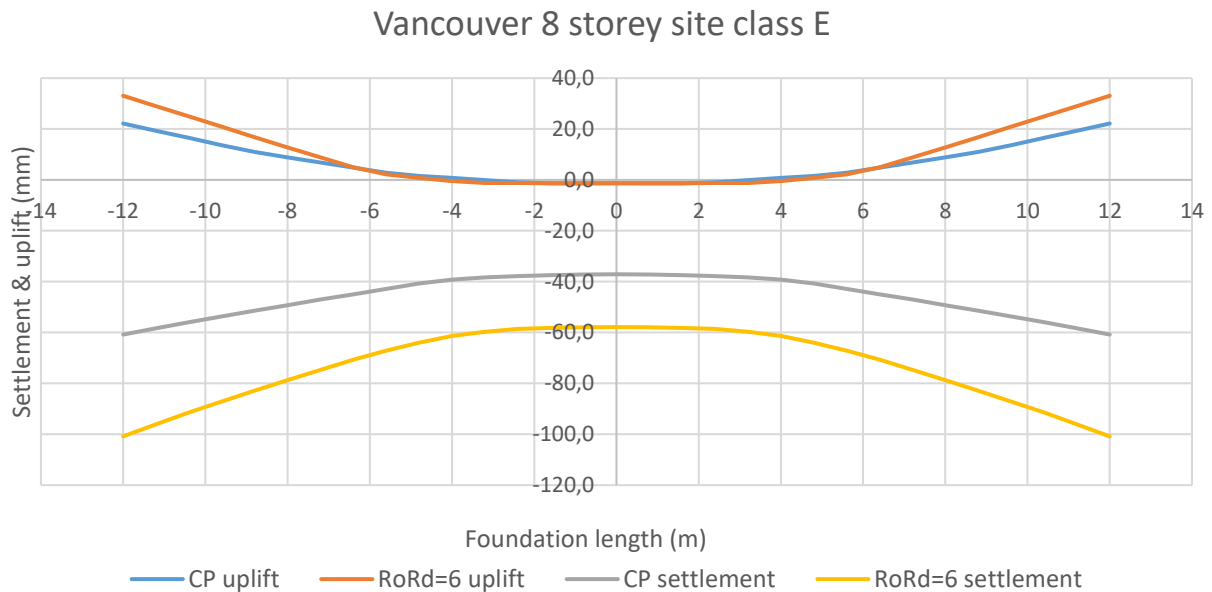


Figure 5-15 Foundation settlement and uplift for an 8-storey building on site class E

5.1.9 Footing rotation

To accurately estimate the real inter-storey drift concerning foundation sizing and soil-structure interaction, it is imperative to incorporate foundation rotation into our calculations, as outlined in section 2.2.1. As indicated in Table 5-9, the initial foundation rotation estimation θ_{design} appears highly conservative in compare with the rotation θ_{analysis} which is extracted from dynamic analysis. The ratio of $\theta_{\text{analysis}} / \theta_{\text{design}}$ for both the 3 and 8-storey buildings, situated on class C site, ranges from 0.04 to 0.16. Conversely, for the same building located on class E site, this ratio varies from 0.07 to 0.35. This suggests that the initial rotation estimation for Class E site is less conservative compared Class C site.

Table 5-9 Foundation rotation on soil class C and E

Structure	Site Class	Foundation type	Rotation		$\theta_{\text{analysis}} / \theta_{\text{design}}$
			θ_{analysis}	θ_{design}	
			(Rad)	(Rad)	
Vancouver 3 storey	C	CP	0.0003	0.0075	0.04
		NCF	0.0024	0.0154	0.16
	E	CP	0.0012	0.0155	0.07
		NCF	0.0022	0.0171	0.13
Vancouver 8 storey	C	CP	0.0006	0.0143	0.04
		NCF	0.0019	0.0135	0.14
	E	CP	0.0035	0.0133	0.26
		NCF	0.0056	0.0161	0.35

5.1.10 Evaluation of the response of outer beam segments, braces and columns

In eccentrically braced frames, it is necessary to assess the response of other structural elements induced by yielding and strain-hardened link. As mentioned earlier, while the link yields and dissipates earthquake energy, the other sections of the structure should ideally exhibit elastic response.

The 8-storey building on Class C site having a CP foundation was chosen to examine the response of outer frame members. The numerical model for this building was analyzed using two accelerograms, VC1 and VC20, which are among the five motions with the highest accelerations. The results for moment and axial forces for each element were extracted from the model and the sections were evaluated under the combined action of moment and axial forces.

As can be seen in Table 5-10, for the accelerogram VC1, the proportion of the moment transmitted from the link to the beam outside of the link ranges between 0.57 and 0.75, whereas the moment passed to the diagonals ranges from 0.25 to 0.43.

Table 5-10 The proportion of the moment transferred by the link to other sections (VC1)

structure element	Floor 8		Floor 7		Floor 6		Floor 5	
	M (kN.m)	ratio	M (kN.m)	ratio	M (kN.m)	ratio	M (kN.m)	ratio
link	121	1	196	1	214	1	240	1
beam outside	79	0.65	119	0.57	158	0.73	175	0.72
diagonal	43	0.35	88	0.43	59	0.27	70	0.28
	Floor 4		Floor 3		Floor 2		Floor 1	
link	285	1	313	1	341	1	358	1
beam outside	208	0.71	236	0.74	260	0.75	266	0.74
diagonal	83	0.29	84	0.26	88	0.25	92	0.26

Under the accelerogram VC20, the ratio of the moment conveyed from the link to the beam outside of the link varies between 0.53 and 0.75, while the moment transferred to the diagonals fluctuates within the range of 0.25 to 0.47.

Table 5-11 The proportion of the moment transferred by the link to other sections (VC20)

structure element	Floor 8		Floor 7		Floor 6		Floor 5	
	M (kN.m)	ratio	M (kN.m)	ratio	M (kN.m)	ratio	M (kN.m)	ratio
link	118	1.00	223	1.00	331	1.00	363	1.00
beam outside	78	0.66	126	0.53	251	0.74	258	0.70
diagonal	41	0.34	112	0.47	88	0.26	108	0.30
	Floor 4		Floor 3		Floor 2		Floor 1	
link	379	1.00	390	1.00	483	1.00	521	1.00
beam outside	284	0.72	300	0.74	382	0.75	406	0.74
diagonal	112	0.28	105	0.26	127	0.25	143	0.26

5.1.10.1 Beam outside of the ductile link

Table 5-12 shows the results for outer beam segment during the VC20 ground motion. It is evident that, in the 5th, 6th, and 7th storeys, both tension and compression interactions exceed 1. To obtain more accurate results and decide whether to maintain or alter the sections, it would be advisable to consider the average of the five highest analysis outcomes.

Table 5-12 Interaction of moment and axial force for beams outside of the link (VC1)

Floor	Beams outside of the link	M _r	T _r	M _{rc}	T _{rc}	Interaction in compression	M _{rt}	T _{rt}	Interaction in tension
		(kN.m)	(kN)	(kN.m)	(kN)		(kN.m)	(kN)	
8	W310X28.3	156	1382	-79	-473	0.92	68	600	0.87
7	W310X32.7	185	1609	-119	-686	1.16	135	657	1.14
6	W460X52	420	2560	-158	-776	0.71	216	861	0.85
5	W460X52	420	2560	-175	-839	0.78	221	949	0.90
4	W460X68	574	3353	-208	-1002	0.69	262	1135	0.80
3	W460X82	708	4043	-236	-1115	0.64	294	1222	0.72
2	W460X89	778	4389	-260	-1184	0.63	290	1235	0.65
1	W460X106	920	5198	-266	-1116	0.52	320	1206	0.58

Table 5-13 Interaction of moment and axial force for beams outside of the link (VC20)

Floor	Beams outside of the link	M _r	T _r	M _{rc}	T _{rc}	Interaction in compression	M _{rt}	T _{rt}	Interaction in tension
		(kN.m)	(kN)	(kN.m)	(kN)		(kN.m)	(kN)	
8	W310X28.3	156	1382	-78	-404	0.85	90	461	0.91
7	W310X32.7	185	1609	-126	-781	1.25	141	803	1.26
6	W460X52	420	2560	-251	-1148	1.09	266	1154	1.09
5	W460X52	420	2560	-258	-1198	1.13	269	1309	1.15
4	W460X68	574	3353	-284	-1355	0.94	319	1397	0.97
3	W460X82	708	4043	-300	-1409	0.81	364	1449	0.87
2	W460X89	778	4389	-382	-1710	1.01	390	1642	0.88
1	W460X106	920	5198	-406	-1656	0.79	396	1530	0.73

5.1.10.2 Braces

Diagonals of EBFs with a fixed connexion between the brace and the beam usually carry axial load and a portion of the link end moment. Consequently, it becomes necessary to consider the interaction between moment and axial force in the design of the brace to determine the nature of their response (elastic or not).

As shown in Table 5-14, the interaction of moment and axial force for the VC1 accelerogram remains below 1 for all sections except for the 7th floor. However, the analysis results for the VC20 accelerogram indicate that interaction values exceeded one for the 2nd, 3rd, 5th, 6th, and 7th floors. For enhanced accuracy, it is recommended to consider the average of the five highest analysis outcomes when deciding whether to retain or modify the sections.

Table 5-14 Interaction of moment and axial force for braces (VC1)

Level	Floor 8	Floor 7	Floor 6	Floor 5
Brace	HSS 178x178x7.9	HSS 203x203x7.9	HSS 229x229x7.9	HSS 229x229x9.5
M_f (kN.m)	43	88	59	70
N_f (kN)	473	996	1265	1401
Max interaction	0.68	1.08	0.95	0.90
Level	Floor 4	Floor 3	Floor 2	Floor 1
Brace	HSS 229x229x13	HSS 229x229x13	HSS 229x229x13	HSS 229x229x16
M_f (kN.m)	83	84	88	92
N_f (kN)	1655	1782	1796	1883
Max interaction	0.83	0.88	0.90	0.78

Table 5-15 Interaction of moment and axial force for braces (VC20)

Level	Floor 8	Floor 7	Floor 6	Floor 5
Brace	HSS 178x178x7.9	HSS 203x203x7.9	HSS 229x229x7.9	HSS 229x229x9.5
Mf (kN.m)	41	112	88	108
Nf (kN)	692	1208	1673	1883
Max interaction	0.88	1.31	1.26	1.22
Level	Floor 4	Floor 3	Floor 2	Floor 1
Brace	HSS 229x229x13	HSS 229x229x13	HSS 229x229x13	HSS 229x229x16
Mf (kN.m)	112	105	127	143
Nf (kN)	2036	2181	2434	2378
Max interaction	1.00	1.06	1.21	0.99

5.1.10.3 Column

The moment at the end of the column on the first level was assessed and compared with $M_{py}=Z_y*f_y$. During the design phase, it was assumed that there would be an additional 0.2 M_{py} in conjunction with the axial force. For an 8-storey building on site Class C, the column section for the first floor is W360X382. The interaction value for both VC1 and VC20 is less than 1, with M_{fy} being 0.19 M_{py} for VC1 and 0.24 M_{py} for VC20, respectively.

Table 5-16 Column's first-level moment and axial force in Vancouver's 8-storey building on site Class C

Design assumption	$M_{py}=Z_y*.345$	1390.35
	0.2 M_{py} (kN.m)	278
	Axial force (kN)	9957
	Interaction	0.9
VC1	M_{fy} (kN.m)	258
	Axial force (kN)	9253
	Interaction	0.82
VC20	M_{fy} (kN.m)	334
	Axial force (kN)	11486
	Interaction	0.99

5.2 Results for Montréal

5.2.1 General comments

To examine the models, 11 calibrated simulated accelerograms from Atkinson's database were utilized for both a 3-storey and an 8-storey building located in Montreal, considering the distance-magnitude scenario. The analysis involved three base stability conditions: fixed base, capacity-protected foundation (CP), and a foundation without any capacity factor consideration (NCF). Additionally, two soil types, namely Site C and Site E, were taken into consideration, resulting in a total of 132 analyses. For each scenario, the average of the five highest responses was selected from the results obtained through the analysis using the 11 sets of accelerograms. It's worth mentioning that presently there are no reliable simulated accelerograms for Montreal, and the results reported in this section are for information only.

5.2.2 Period of vibration

In every scenario, the structures exhibit longer periods when accounting for soil-structure interaction. The most significant increase was observed in the case of the Montreal 8-storey structure on Class E site, showing an approximate 35% rise. Conversely, the smallest increase of about 4% was observed for the Montreal 8-storey structure on Class C site.

Table 5-17 Period change for different types of soil and foundations (Montreal)

Structure	Site Class	Foundation type	$T_{1,SSI}$ (s)	$T_{1,FIX}$ (s)	$T_{1,SSI}/T_{1,FIX}$
Montreal 3 storey	C	CP	0.68	0.65	1.05
		NCF	0.70		1.08
	E	CP	0.68	0.62	1.10
		NCF	0.69		1.11
Montreal 8 storey	C	CP	1.96	1.88	1.04
		NCF	2.00		1.06
	E	CP	2.16	1.64	1.32
		NCF	2.21		1.35

5.2.3 Lateral displacement

For 3-storey buildings situated in Montreal on Class C and E sites, the foundations designed without a capacity-protected factor exhibited the highest inter-storey drift. However, these drifts were smaller compared to buildings with a capacity-protected (CP) foundation, indicating a higher rotation and additional drift along the building's height. The maximum inter-storey drift occurred at the second level for both soil types, measuring 0.65% for soil type C and 0.91% for soil type E. In all cases, the inter-storey drifts remained below the NBCC design limit.

Table 5-18 Inter-storey drift index for 3-storey frame site Class C and E

Structure	Site Class	Foundation type	Storey		
			1	2	3
Montreal 3 storey	C	Fixed base	0.61%	0.62%	0.27%
		CP	0.62%	0.65%	0.30%
		NCF	0.62%	0.65%	0.40%
	E	Fixed base	0.66%	0.76%	0.36%
		CP	0.84%	0.90%	0.43%
		NCF	0.80%	0.91%	0.47%

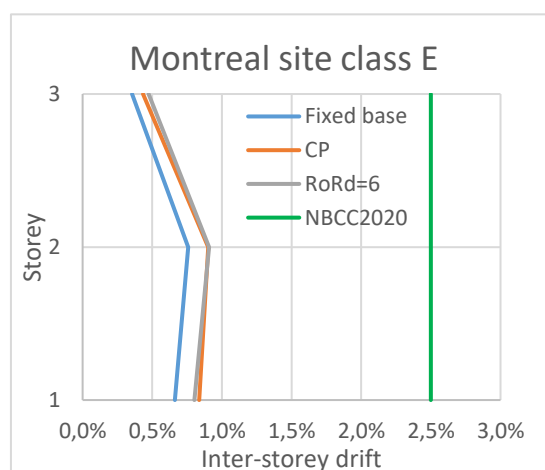
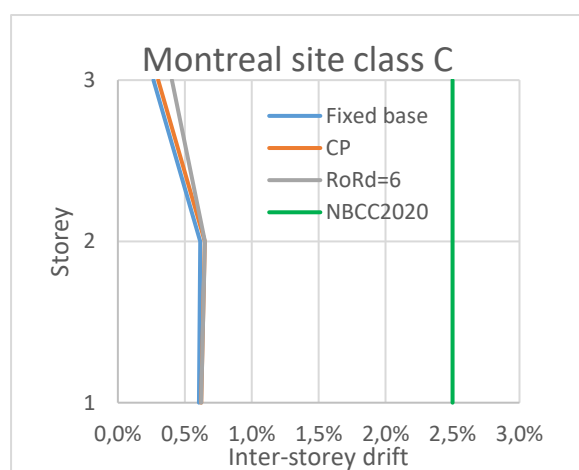


Figure 5-16 Inter-storey drift for Montreal 3-storey frame, Class C and E sites

For both class sites, 8-storey buildings with foundations designed without a capacity-protected factor, exhibited the most notable inter-storey drift. However, these drifts still remained below the NBCC design limit of 2.5% h. The trend observed in Table 5-19 indicates that inter-storey drift increases as the foundation size decreases. Specifically, the 8-storey building on soil type C site exhibits the highest inter-storey drift at the 6th level, approximately 1.02%, while it reaches 1.39% at the 6th level when the soil type changes to E.

Table 5-19 Inter-storey drift percentage for 8-storey structure site class C and E (Montreal).

Structure	Site Class	Foundation type	Storey							
			1	2	3	4	5	6	7	8
Montreal 8 storey	C	Fixed base	0.58%	0.49%	0.50%	0.55%	0.72%	0.97%	0.78%	0.67%
		CP	0.43%	0.56%	0.52%	0.58%	0.73%	0.99%	0.81%	0.69%
		NCF	0.44%	0.57%	0.51%	0.58%	0.73%	1.02%	0.83%	0.70%
	E	Fixed base	0.80%	0.72%	0.73%	0.89%	1.10%	1.39%	1.24%	0.92%
		CP	0.64%	0.89%	0.88%	0.96%	1.13%	1.38%	1.16%	0.91%
		NCF	0.68%	0.91%	0.90%	0.97%	1.15%	1.35%	1.18%	0.91%

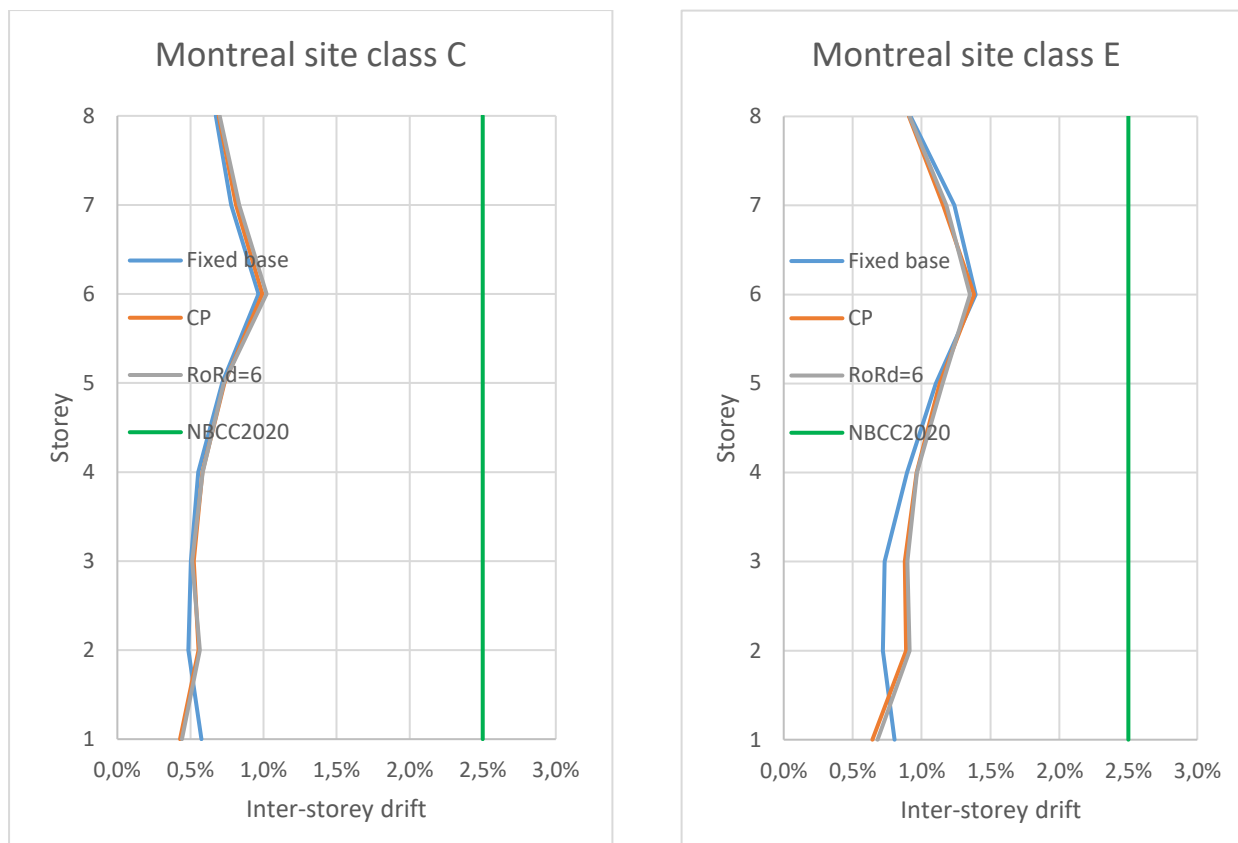


Figure 5-17 Inter-storey drift for Montreal 8-storey site class C and E

5.2.4 Overturning moment

To assess the impact of foundation size on overturning moment and energy dissipation, various scenarios were examined to understand how soil-structure interaction mitigates the overturning moment. The results indicate a reduction in the rate of M_{fSSI}/M_{fFB} by 1% to 9% as foundation size decreases. Notably, in 8-storey scenarios situated on site class E, soil-structure interaction is more prominent, with $M_{RdRo=6}$ being approximately 9% less than M_{fFB} . Similarly, for site class C scenarios, this difference is around 8%.

Table 5-20 Overturning moment for different type of foundations (Montreal).

Structure	Site Class	Foundation type	Design	Fixed-base	Full model	M_d/M_{fFB}	M_{fSSI}/M_{fFB}
			M_d (kN.m)	M_{fFB} (kN.m)	M_{fSSI} (kN.m)		
Montreal 3 storey	C	CP	19321	17527	17284	1.10	0.99
		NCF	8018	17527	15411	0.46	0.88
	E	CP	23347	23240	22706	1.00	0.98
		NCF	11421	23240	22386	0.49	0.96
Montreal 8 storey	C	CP	64943	51299	49139	1.27	0.96
		NCF	27412	51299	47347	0.53	0.92
	E	CP	90823	85062	81235	1.07	0.96
		NCF	47234	85062	77414	0.56	0.91

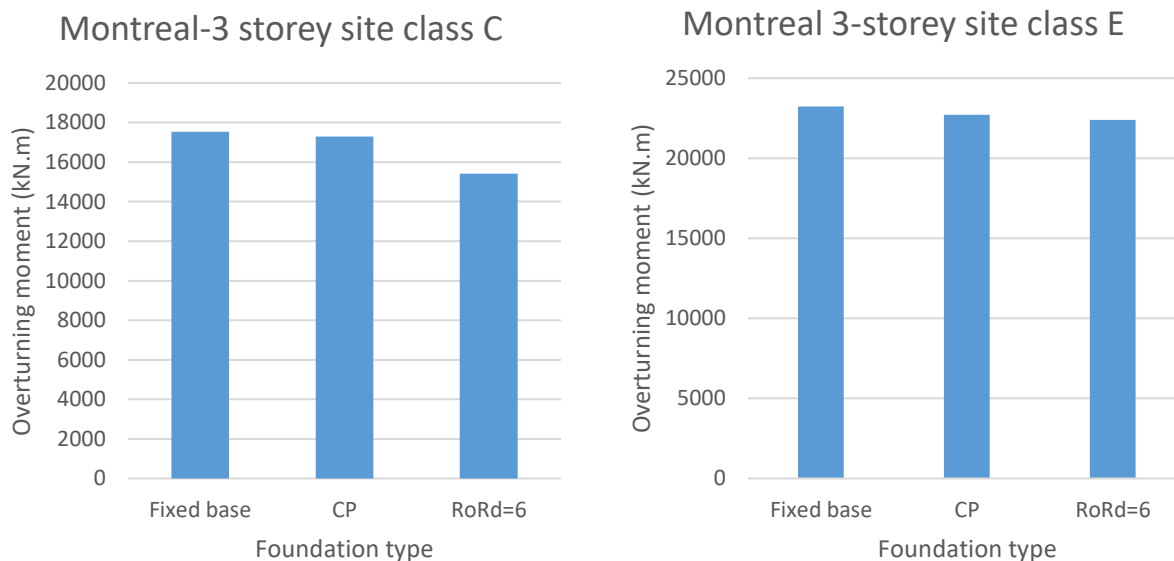


Figure 5-18 Comparison the magnitude of overturning moment for 3-storey on site class C and E

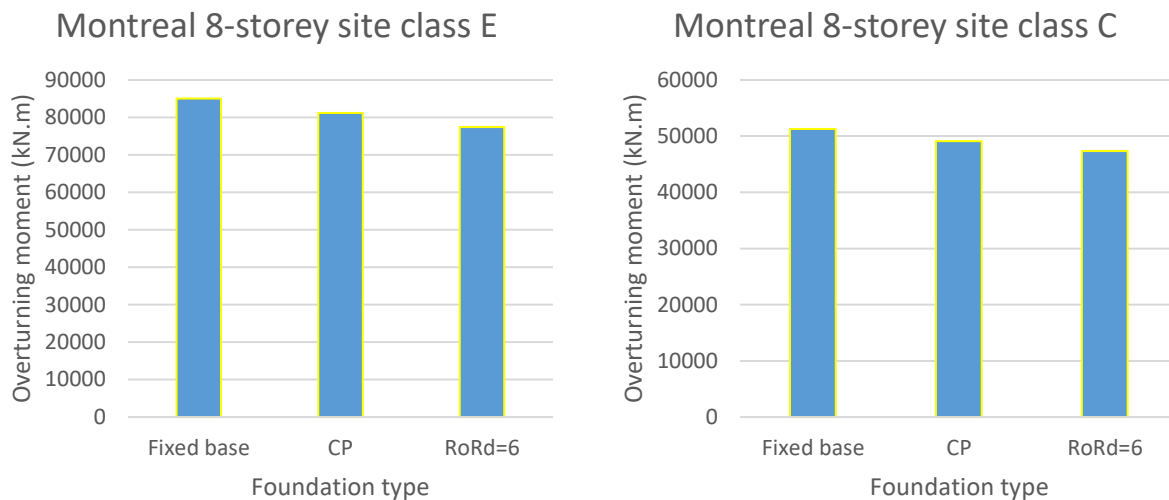


Figure 5-19 Comparison the magnitude of overturning moment for 8-storey on site class C and E

5.2.5 Link inelastic rotation

The inelastic rotation for three-storey structures in Montreal ranges from 0.003 rad to 0.06 rad. The largest value, of about 0.06 rad, is observed on the second floor of the CP foundation at site class E, while the minimum is about 0.003 rad for the third floor of the NCF foundation at site class C. It can be observed that, in most cases, the Ω factor does not reach the CSA S16 value of 1.3. This could be interpreted as an indication that the accelerograms might not have been strong enough to push the structure to its maximum ductility limits.

Table 5-21 Inelastic rotations for an 3-storey building in Montreal on site class C and E

Structure	Site Class	Foundation type	Storey γ (rad)		
			1	2	3
Montreal 8 storey	C	Fixed base	0.051	0.048	0.003
		CP	0.049	0.049	0.003
		NCF	0.036	0.040	0.003
	E	Fixed base	0.053	0.054	0.007
		CP	0.059	0.060	0.008
		NCF	0.053	0.056	0.006

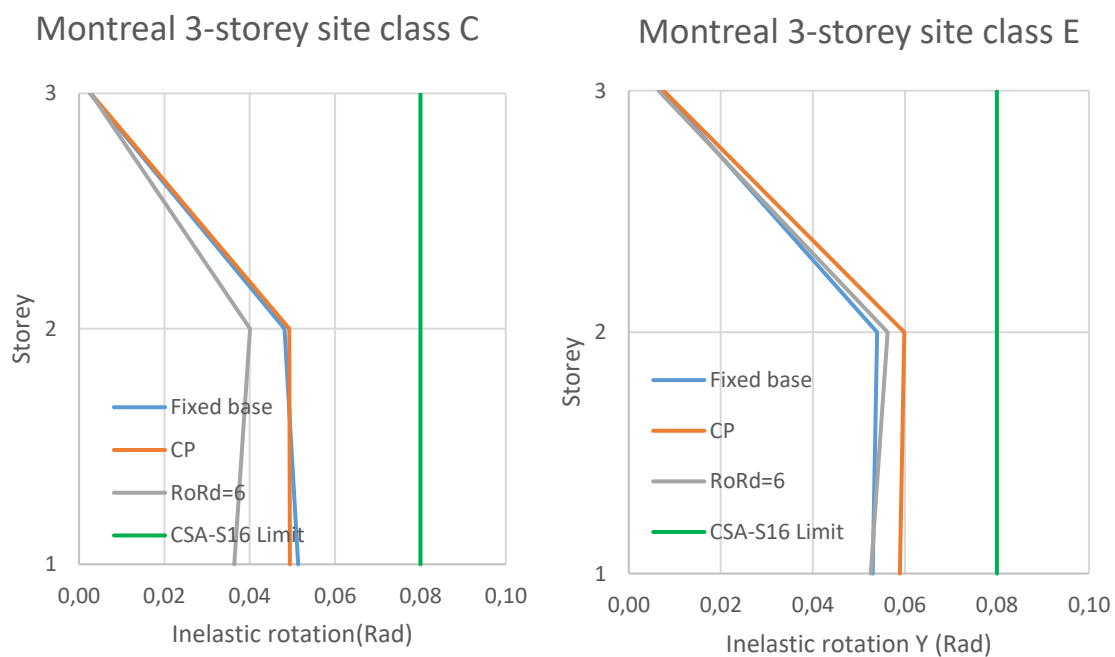


Figure 5-20 Inelastic rotations for 3-storey building in Montreal on site class C and E

The inelastic rotation of 8-storey structures in Montreal varies between 0.013 rad and 0.113 rad. The maximum value, around 0.113 radians, is noted on the 6th floor of the fixed base foundation type at site class E, whereas the minimum is approximately 0.013 radians for the 8th floor of the fixed base foundation type at site class C.

Table 5-22 Inelastic rotations for 8-storey building in Montreal on site class C and E

Structure	Site Class	Foundation type	Storey γ (rad)							
			1	2	3	4	5	6	7	8
Montreal 8 storey	C	Fixed base	0.041	0.032	0.028	0.021	0.034	0.067	0.036	0.013
		CP	0.022	0.036	0.028	0.018	0.037	0.071	0.037	0.014
		NCF	0.022	0.035	0.024	0.016	0.032	0.071	0.035	0.014
	E	Fixed base	0.067	0.050	0.049	0.058	0.072	0.113	0.075	0.041
		CP	0.032	0.055	0.051	0.052	0.072	0.112	0.067	0.041
		NCF	0.033	0.054	0.045	0.044	0.068	0.104	0.066	0.043

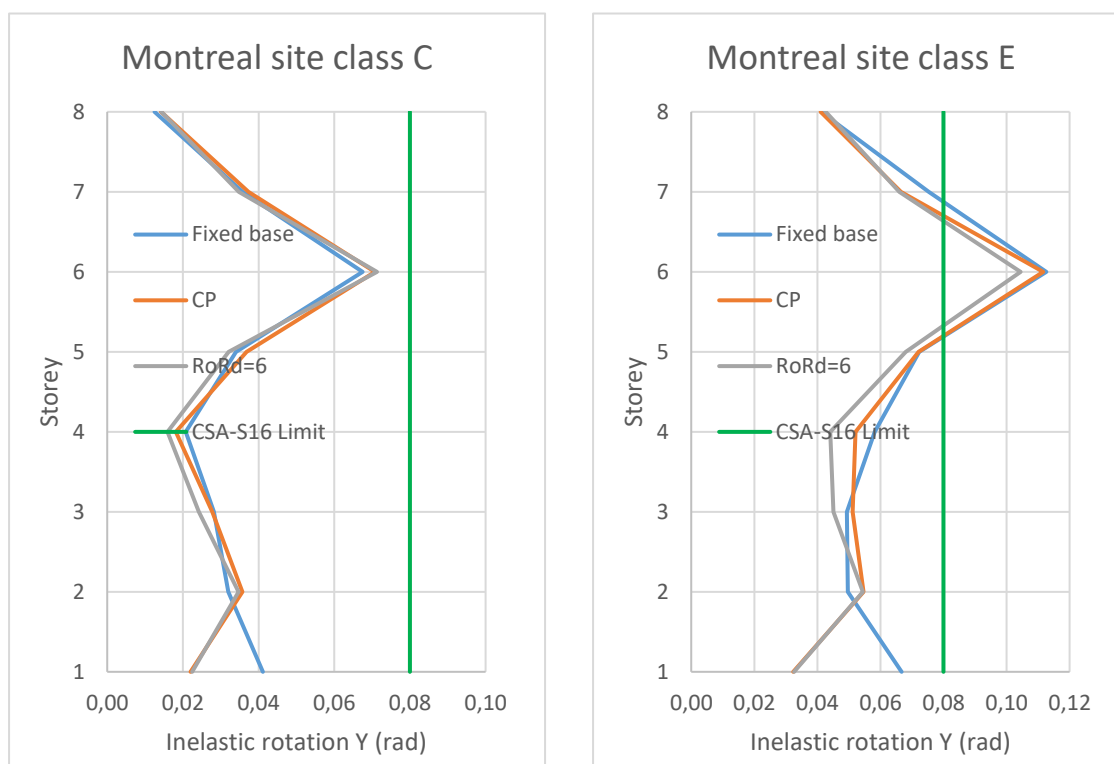


Figure 5-21 Inelastic rotations for 8-storey building in Montreal on site class C and E

5.2.6 Shear yielding capacity of ductile links

For a three-storey building located on soil designated as type C, the highest Ω ratio is seen on the second-floor level for the CP foundation, measuring 1.21, while it is about 1.25 for fixed base model situated on Class E site.

Table 5-23 The ratio of Ω for 3 storey structure on site class C and E

Structure	Site Class	Foundation type	Storey Vmax/Vp (Ω)		
			1	2	3
Montreal 8 storey	C	Fixed base	1.19	1.20	0.91
		CP	1.18	1.21	0.93
		NCF	1.15	1.17	0.91
	E	Fixed base	1.24	1.25	1.04
		CP	1.24	1.25	1.04
		NCF	1.21	1.24	1.04

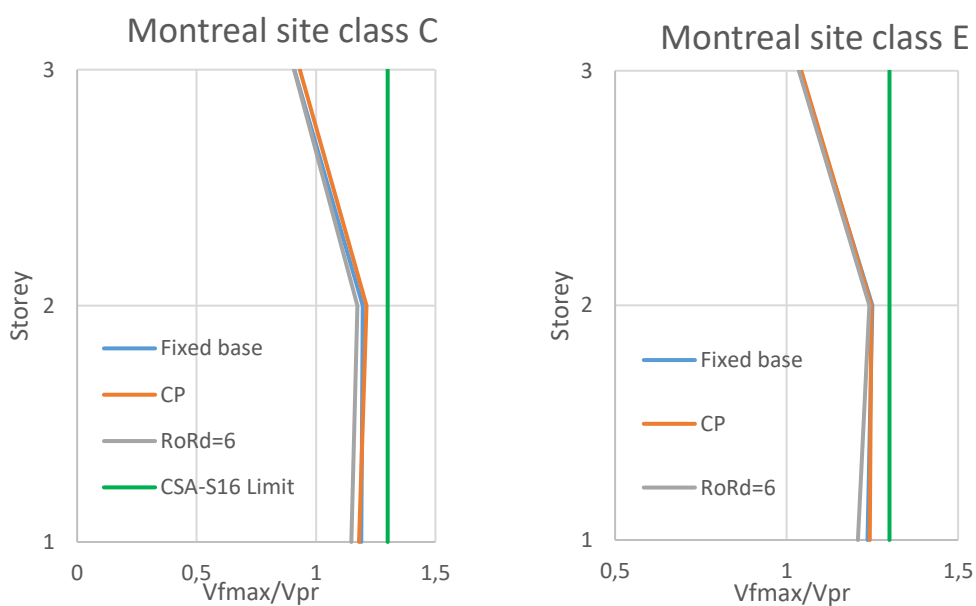


Figure 5-22 The ratio of V_{max}/V_{pr} for 3-storey structures on site class C and E

For the 8-storey building situated on Class C site, the greatest Ω ratio occurs at the 6th floor level in the NCF model, registering at 1.21. Meanwhile, on site class E, the Ω ratio for the 7th floor in the CP model is approximately 1.34. It is noteworthy that, in many instances Table 5-24, the Ω factor does not attain the CSA S16 threshold of 1.3. This observation suggests that the accelerograms may not have exerted sufficient force to push the structure to its maximum ductility limits.

Table 5-24 The ratio of Ω for 8 storey structure site class C and E

Structure	Site Class	Foundation type	Storey Vz/Vp (Ω)							
			1	2	3	4	5	6	7	8
Montreal 8 storey	C	Fixed base	1.17	1.12	1.08	1.09	1.15	1.23	1.11	1.05
		CP	1.10	1.16	1.09	1.09	1.17	1.24	1.13	1.06
		NCF	1.09	1.15	1.07	1.07	1.16	1.26	1.13	1.06
	E	Fixed base	1.24	1.19	1.18	1.18	1.26	1.11	1.33	1.17
		CP	1.12	1.19	1.15	1.15	1.23	1.11	1.34	1.18
		NCF	1.12	1.18	1.14	1.13	1.21	1.11	1.32	1.15

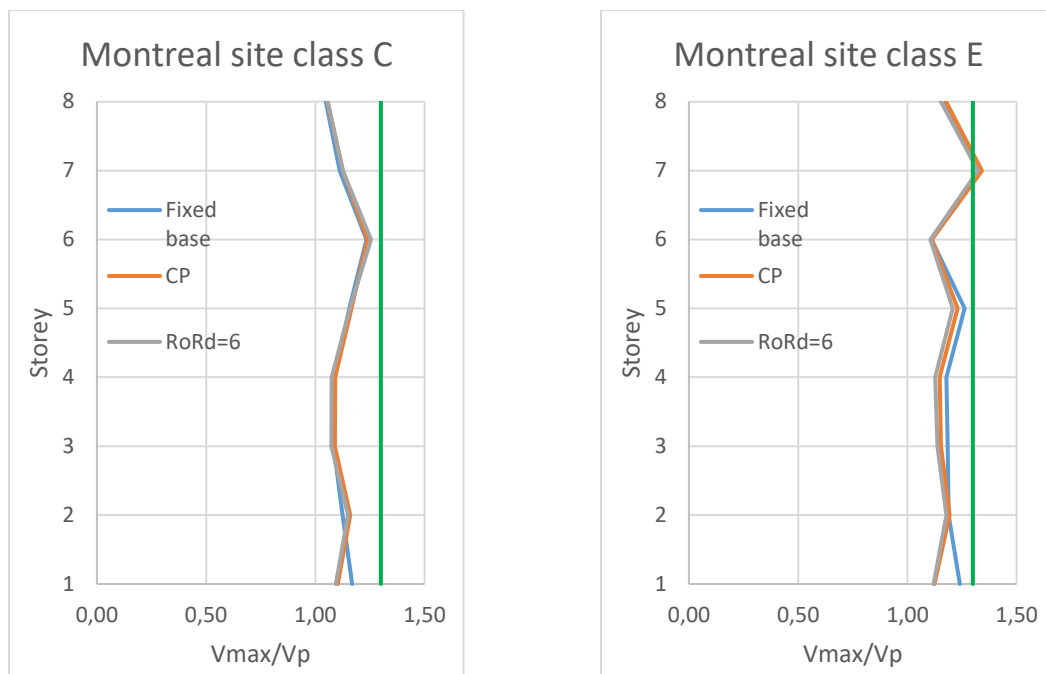


Figure 5-23 The ratio of $V_{\max}/V_{pr}$ for 8-storey structures on Site Class C and E

5.2.7 Settlements and uplift

In the 3-storey building situated on soil class C as shown in Figure 5-24, the NCF ($R_d R_o=6$) foundation type exhibits the highest uplift, measuring 14mm, whereas the CP foundation shows a uplift of around 1.7mm. In terms of settlement, the NCF foundation type experiences the most significant one, reaching -5mm, as opposed to the CP foundation which settles approximately 2.7mm. For the Class E site, the maximum settlement occurs for the NCF foundation type, measuring -21mm, while the CD type of foundation in Figure 5-25 shows a settlement of -18mm.

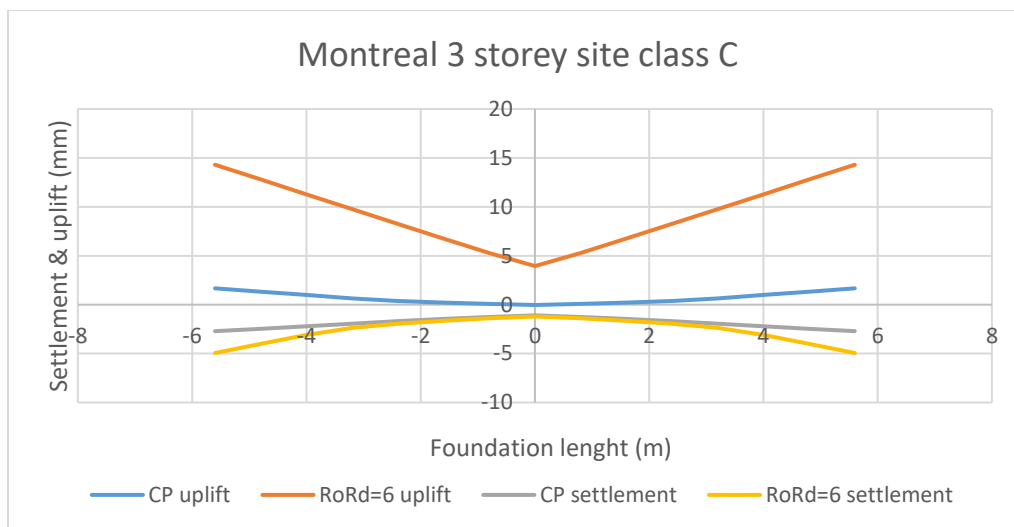


Figure 5-24 Foundation settlement and uplift for an 8-storey building on site class C

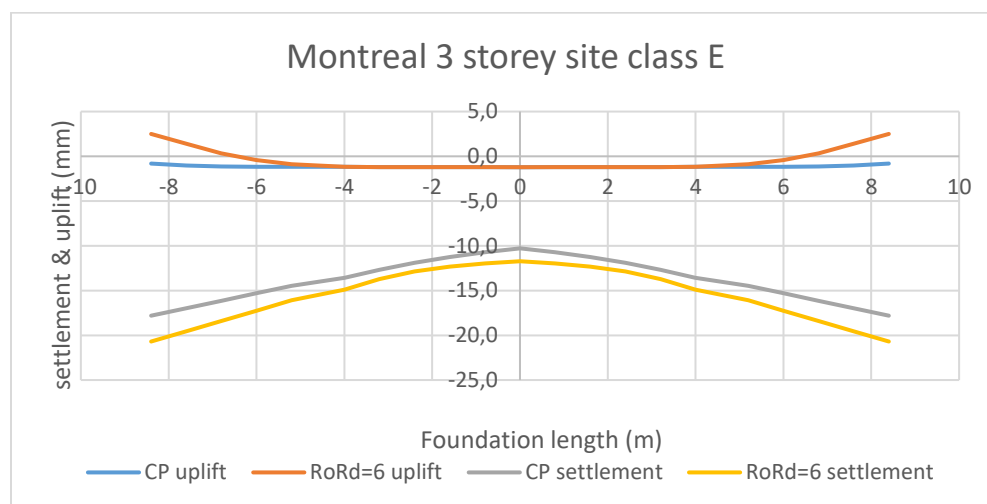


Figure 5-25 Foundation settlement and uplift for a 3-storey building on site class E

As seen in Figure 5-26, for the 8-storey building situated on a Class C site has the largest uplift, measuring 5mm, was recorded for NCF foundation while the CP foundation shows an uplift of about 1mm. NCF foundation type demonstrates the most significant settlement, equal to -5mm, as compared to the CP foundation, which settles approximately -4mm. Furthermore, for an 8-storey building on Class E site, the NCF foundation exhibits the most significant settlement equal to -40 mm, while for the CP foundation, the settlement of about -33mm was recorded (see Figure 5-27).

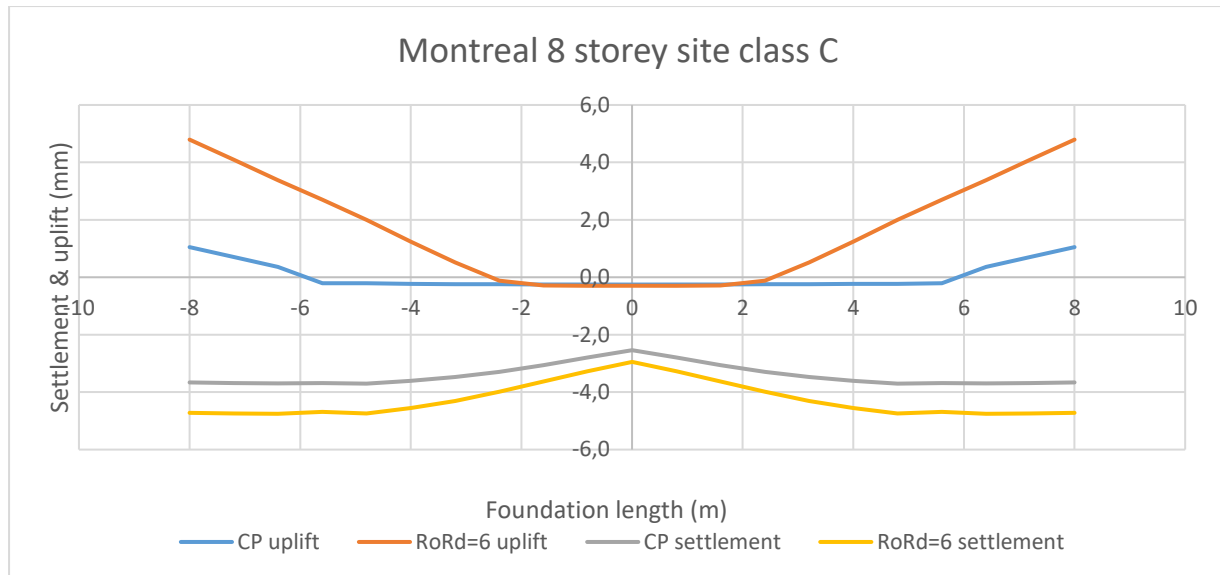


Figure 5-26 Foundation settlement and uplift for an 8-storey building on site class C

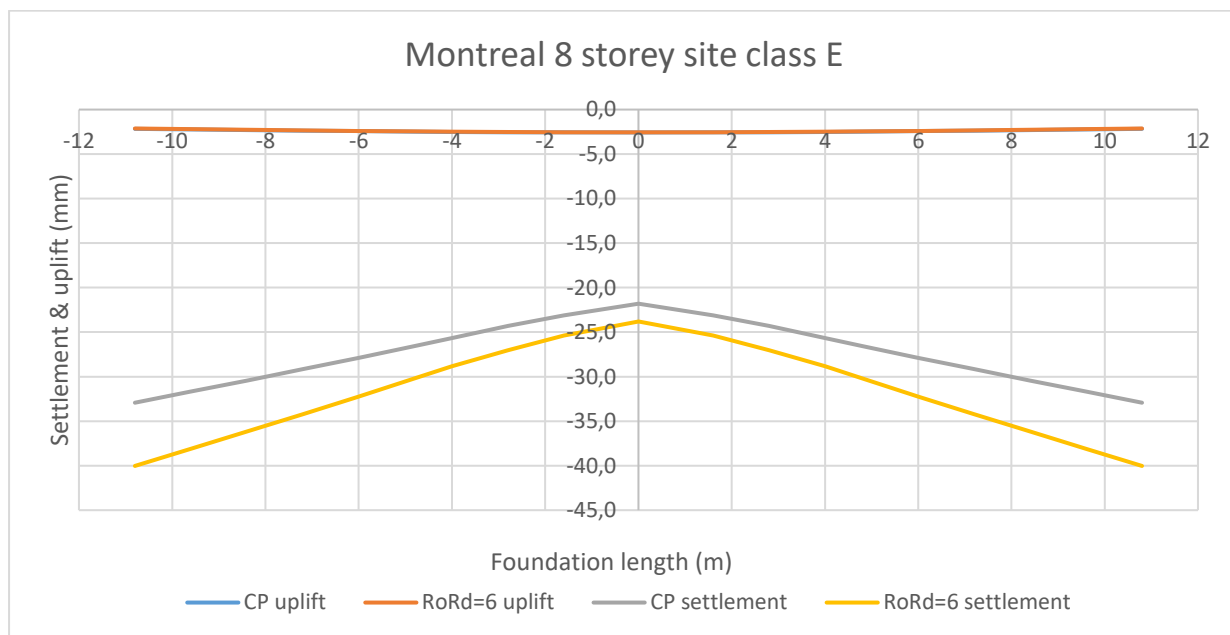


Figure 5-27 Foundation settlement and uplift for an 8-storey building on site class E

CHAPTER 6 CONCLUSION AND RECOMMENDATIONS

This chapter provides a summary of the project and presents the main conclusions and recommendations.

6.1 Summary

Past research demonstrates that foundation movement, specifically rocking, can reduce the demand on the superstructure by dissipating a portion of earthquake energy through soil-structure interaction. This behaviour may enhance building resilience and safety during seismic events. Intentionally designing foundations for controlled movement, known as foundation rocking, may improve in seismic performance. Traditional fixed foundations can directly transmit seismic forces, potentially causing severe damage; however, rocking foundations allow buildings to sway, reducing lateral forces on the superstructure. The CSA A23.3-19 outlines a process that acknowledges this behaviour and presents two options for foundation design. Capacity Protected foundations aim to restrict foundation rocking by employing a design moment equal to or greater than the capacity overturning moment of the SFRS which may results in larger footing dimensions, limiting the degree of rocking. Conversely, procedures for Not-Capacity Protected foundations acknowledge that insufficient foundation dimensions may not support the complete overturning capacity of the SFRS, potentially allowing for additional foundation rotation. This design procedure is primarily drawn from studies on concrete seismic systems thus the specifications and safety factors in the Canadian steel design standard may be inadequate. Therefore, using the same foundation design regulation for steel braced frames may not be adequate.

The foundation design procedure for concrete shear wall systems is based on loads stemming from ductility demands concentrated at the base of the shear walls. Conversely, in braced steel frames, ductility demands is distributed in nature and originate from plasticity mechanisms that develop in all ductile frame elements, resulting in cumulative loads on the foundation which may lead to overly conservative estimations for foundation requirements. This research aims to validate the CSA A23.3-19 standard by evaluating the behaviour of two office buildings one with 3- and another with 8-storey as their foundation sizes are altered. These structures are located in two seismic zones, Montreal and Vancouver, and on-site classes C and E. Three distinct types of bases are under consideration: fixed base, Capacity-Protected (CP), and Non-Capacity-Factored $R_d R_o = 6$ (NCF). The fixed base scenario pertains to conditions where soil-structure interaction does not

impact the overturning moment. Generally, soil-structure interaction tends to reduce the demand on the foundation. Thus, utilizing the fixed base scenario offers a conservative estimation, representing an upper limit for the overturning moment on the foundation. A 2D numerical model was developed using the STKO software (also referred to as The Scientific Toolkit for OpenSees) to analyze steel eccentrically braced frames with integrating soil-structure interaction. Nonlinear elements featuring fiber sections, capable of undergoing plasticity, were employed to model the ductile links, beams, and braces. The section aggregator facilitated the simulation of shear behaviour in the ductile links, enabling the development of shear plasticity within these sections. Elastic elements were used to model the columns, ensuring their elasticity persisted throughout loading and unloading cycles. The simulation of soil-structure interaction utilized the beam on Nonlinear Winkler Foundation (BNWF) substructure method. This method characterized the foundation as an elastic beam, while the soil's attributes were represented using zero-length spring elements, effectively capturing the soil's rocking status, uplift, or settlement. Ultimately, time-history analyses were conducted using ground motion records compatible with design spectra. The outcomes of these analyses were compared with Canadian code requirements to evaluate the code limits and procedures.

The results for the 3- and 8-storey buildings in Vancouver on site Class C shows that CP and NCF foundations are within the limits anticipated by Canadian design procedure. The maximum inter-storey drift for the 3-storey structure was 0.99%, and for the 8-storey structure, it was approximately 1.58% of the building height, which is below the upper limit of 2.5%. The soil stress demand in all cases was below the soil bearing capacity, while the permanent settlement was almost zero. It could be said that for these two prototype buildings, the foundation size could be less than what is presently required in the Canadian code.

The maximum inter-storey drift for the 3-storey structure on Class E site was 1.06%, and for the 8-storey structure, it was approximately 2.0% of the building height. The soil stress demand in all cases remained below the soil bearing capacity. However, the non-capacity factored foundation NCF settlement for the 3 and 8-storey buildings reached -31mm and -101mm, respectively, while the permanent settlement measured about -16mm and -58mm for the 3 and 8-storey NCF foundations, respectively thus exceeding the 25mm limit. Moreover, during the analysis some convergence problems were observed in certain cases which may be related to the weak properties of soil class E. Non-capacity factored foundation for both 8 and 3-storey building on soil class E

did not show adequate response and thus this design approach could not be validated.

It's worth mentioning that at the present time there are no reliable simulated accelerograms for Montreal, and the results obtained for Montreal buildings are provided for information only.

6.2 Conclusions

The primary findings derived from the analyses using nonlinear time history are listed below.

- Eccentrically braced frames with shear links exhibit a higher overstrength Ω factor compared to the standard outlined in CSA S16-19. This research suggest that the structure's ductility might surpass design predictions. When the moment-resistant connection was assumed between the beam and brace, the transfer of moment from ductile link to braces was substantial, approaching the value calculated during the design phase. In line with the findings from previous research, a moment arising from the column continuity was recorded with values exceeding higher $0.2Z_yF_y$ predicted in design.
- A significant reduction in overturning moment demand was observed when the foundation size was reduced. This trend confirms that soil-structure interaction may significantly affect overturning moment and impacts the demand on superstructure sections. The dissipation of energy happens as the foundation rocks, influenced by both its dimensions and the specifications of the soil it is situated on.
- In most cases, the overturning moment transferred from the superstructure to the foundation exceeded the $M_{probable}$, corresponding to the value of the overturning moment calculated assuming that all the links yield simultaneously. This suggests potential overstrength in the link and emphasizes the influence of other parts of the brace in moment transfer.
- As anticipated in design, a foundation rotation occurred due to soil-structure interaction, resulting in additional inter-storey drift. The magnitude of this drift is influenced by both the foundation size and the bearing capacity of the soil. When the size is reduced and the soil has a low bearing capacity, there tends to be a higher level of additional inter-storey drift. In this study, CP and NCF (non-capacity factored foundation) were conceived for two prototype buildings on. For Class C site cases, the inter-storey drift stayed below 2.5% of the structure's height. This suggests that the limitations on foundation size stipulated in the

Canadian code may be conservative. It is possible that the dimensions could be smaller than those required by the CSA A23.3-19 standard for soil class C.

- The results for Class E site were somewhat different: the prototype structure experienced excessive settlement in both CP and non-capacity factored foundations, yet their inter-storey drift remained below 2.5% of the structure's height. Despite adhering to the inter-storey drift limit, there were some convergence issues in the nonlinear time history analysis, particularly concerning the lower frames, possibly associated with the low soil strength. Therefore, reducing the foundation size compared to the one required by CSA A23.3-19 standard could not be validated.

This study focused on eccentrically braced frames for 3-and 8-storey structures positioned on soil classes C and E. To draw broader conclusions, it may be necessary to analyze structures with varying elevations and placed on different site Classes.

6.3 Recommendations

Recommendations for potential improvements to further understand the behavior of shallow foundations in eccentric steel frames are detailed below.

- This study on the foundations of steel eccentrically braced frames is part of a larger research effort exploring how soil-structure interaction affects the seismic design of foundations for steel frames. Similar research has already been done for concentrically braced frames. To better understand the design requirements for steel frame foundations, it is recommended to conduct further research on different Seismic Force Resisting Systems (SFRS), like moment-resisting steel frames, while considering a variety of elevations and site class.
- while the BNWF concept is widely used in the literature to represent soil-structure interaction, exploring an alternative approach that is computationally manageable in terms of complexity and processing time could be beneficial. This would help to validate the results presented in this thesis.
- An experimental test at a reduced scale would be necessary to verify the behavior of a superstructure-foundation-soil interaction that includes eccentrically braced frames.

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APPENDIX A STKO MODELS AND DIAGRAMS

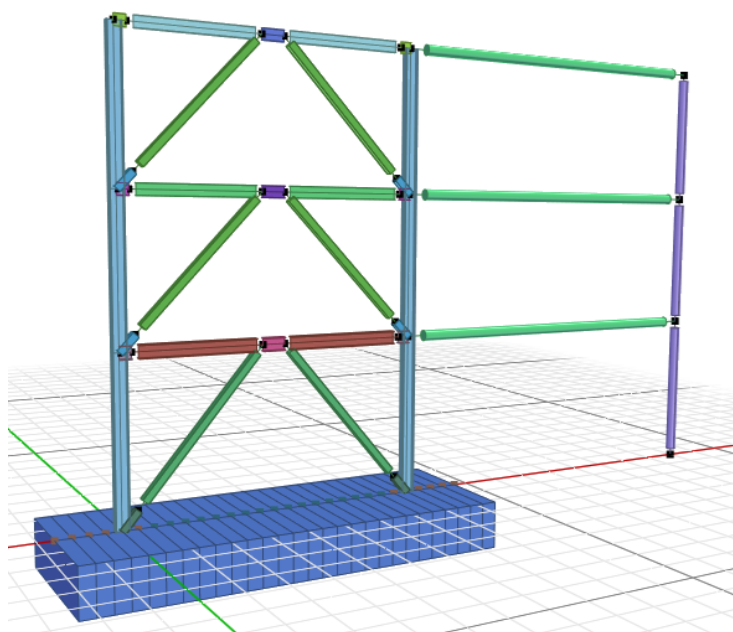


Figure 6-1 Vancouver 3-storey structure on soil class C STKO model

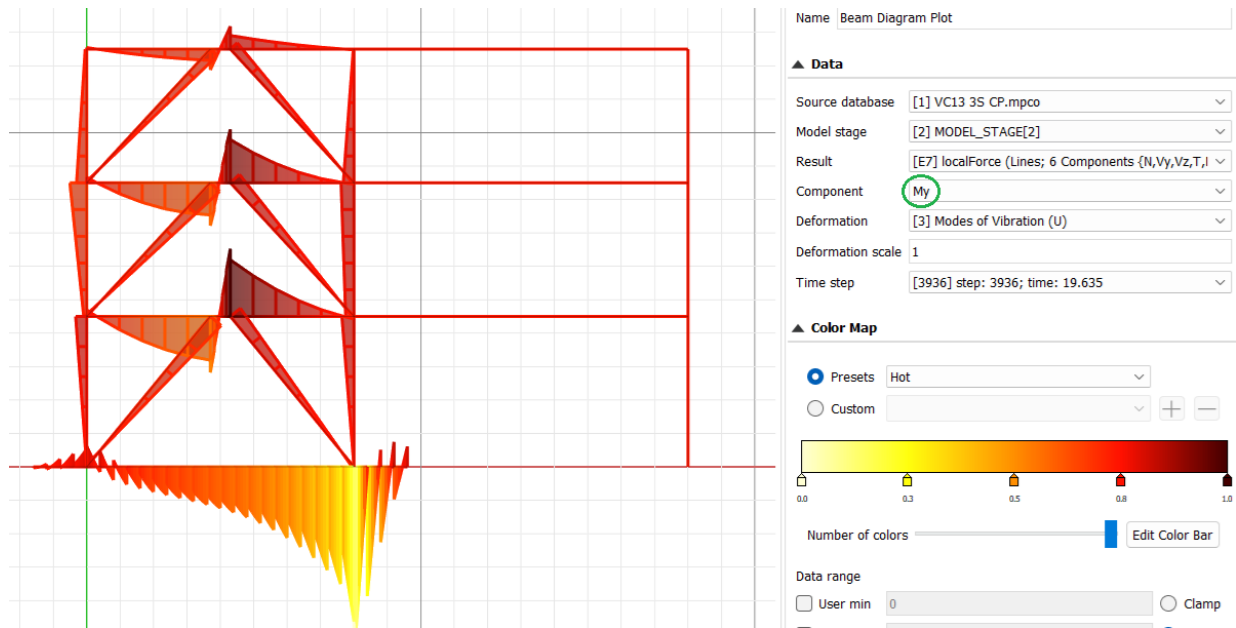


Figure 6-2 Vancouver 3-Storey structure on Soil Class C - A Sample for Moment Diagram

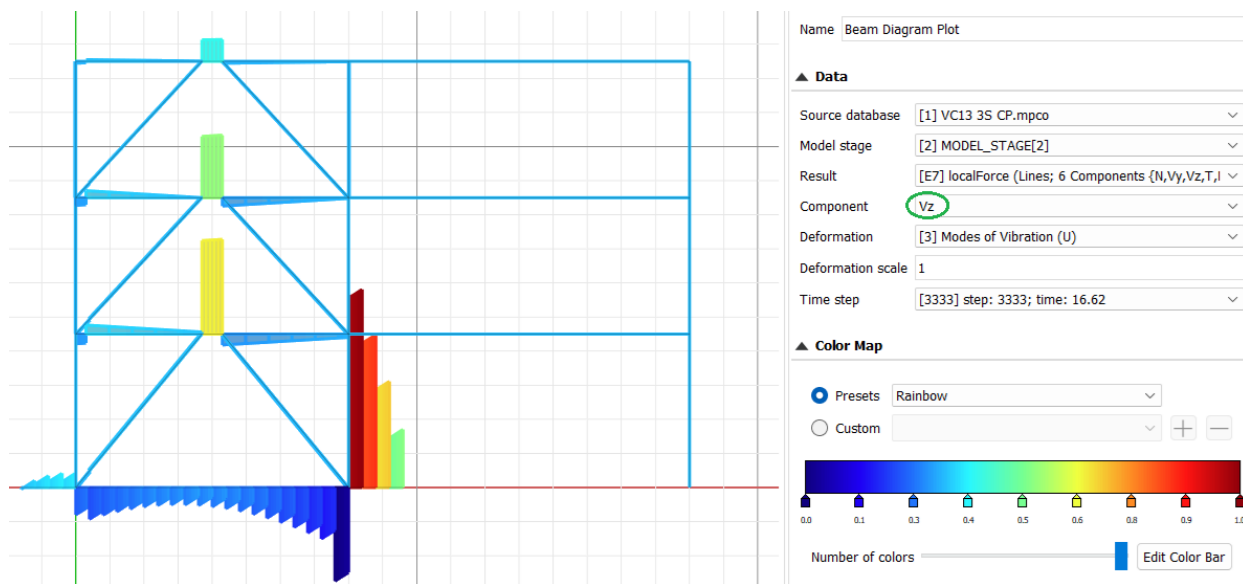


Figure 6-3 Vancouver 3-Storey structure on Soil Class C - A Sample for shear force diagram

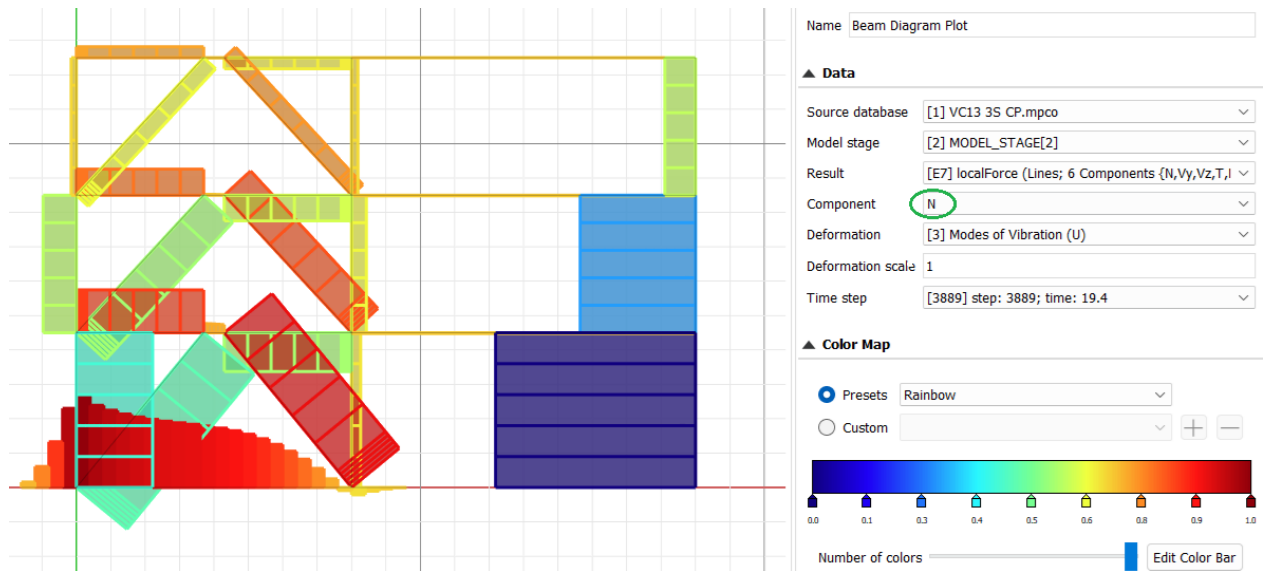


Figure 6-4 Vancouver 3-Storey structure on Soil Class C - A Sample for axial force diagram

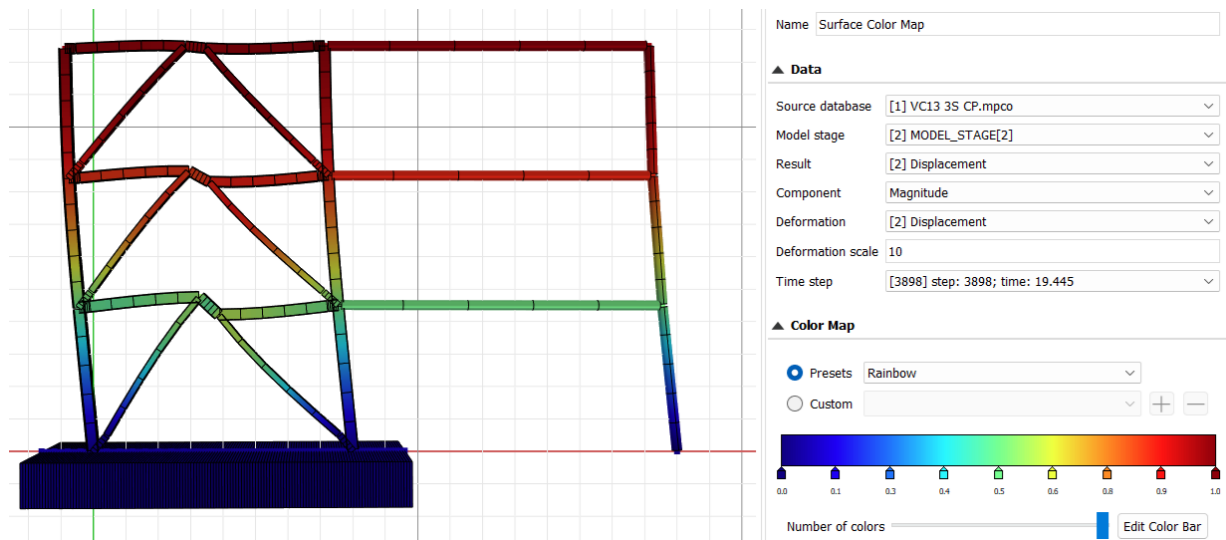


Figure 6-5 Vancouver 3-Storey structure on Soil Class C displacement

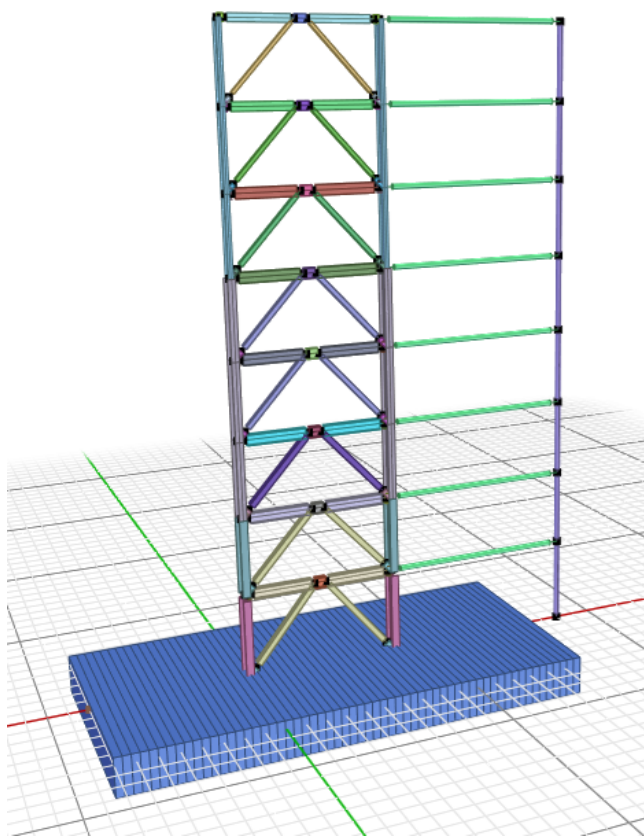


Figure 6-6 Vancouver 8-storey on soil class E STKO model

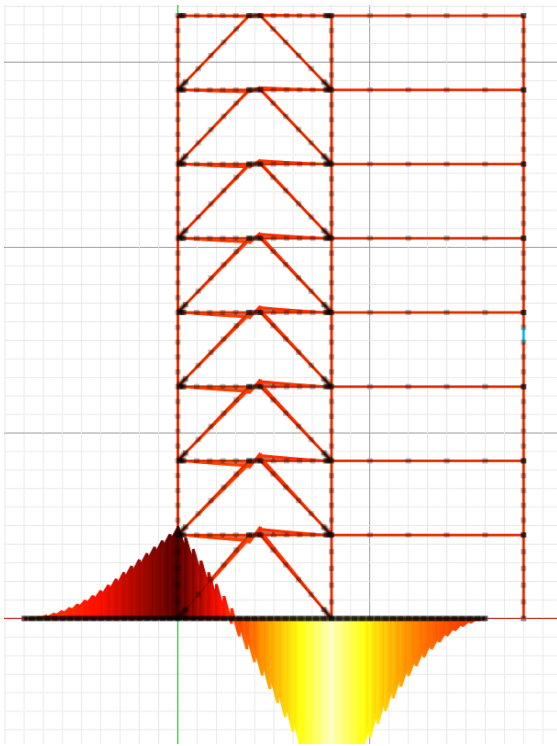


Figure 6-8 Vancouver 8-storey on site soil E
A sample for moment diagram

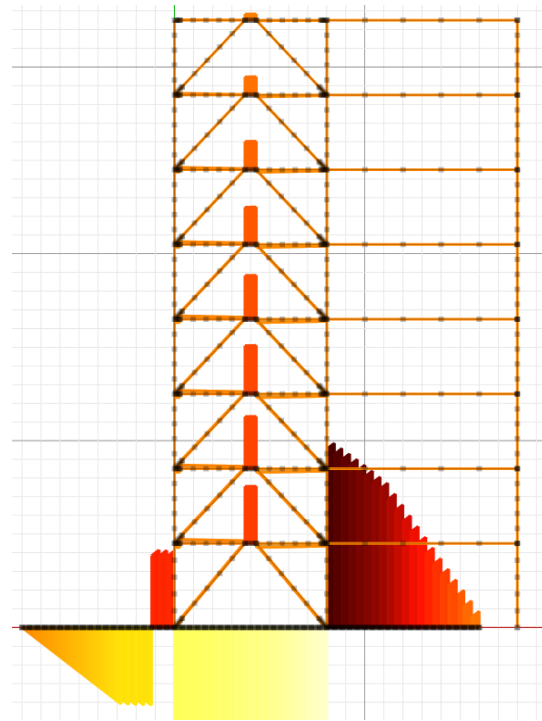


Figure 6-9 Vancouver 8-storey on site soil E

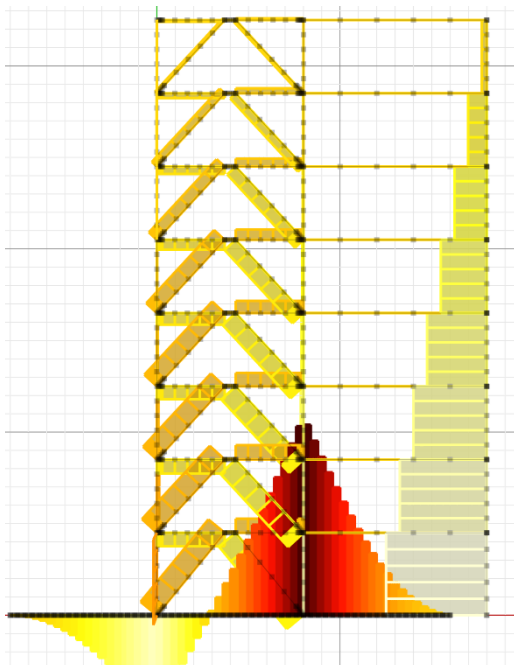


Figure 6-10 Vancouver 8-storey on site soil E

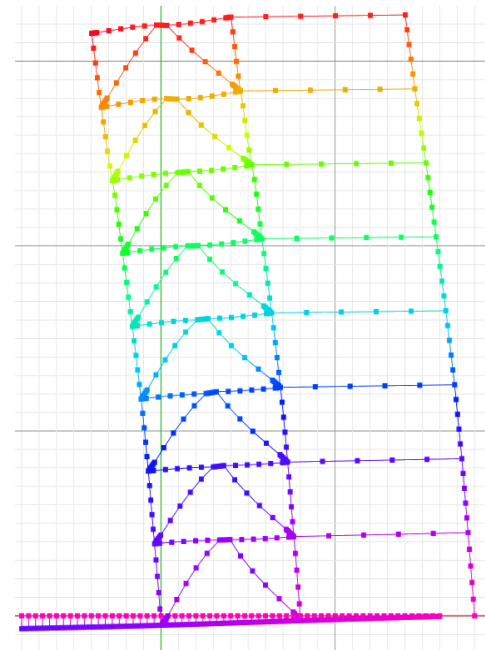


Figure 6-7 Vancouver 8-storey on site soil E

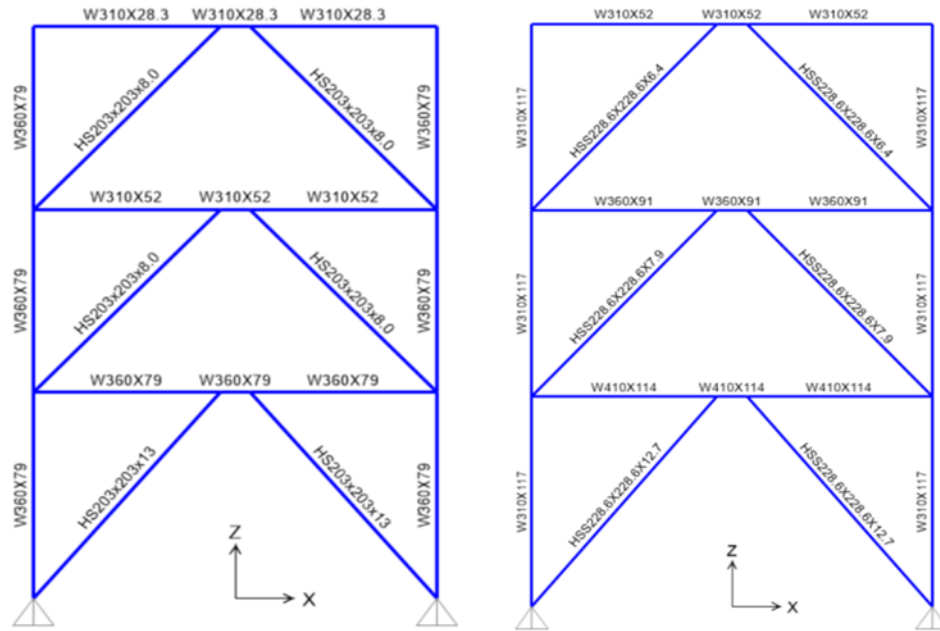


Figure 6-11 Sap2000 models for 3-storey structure soil class C and E

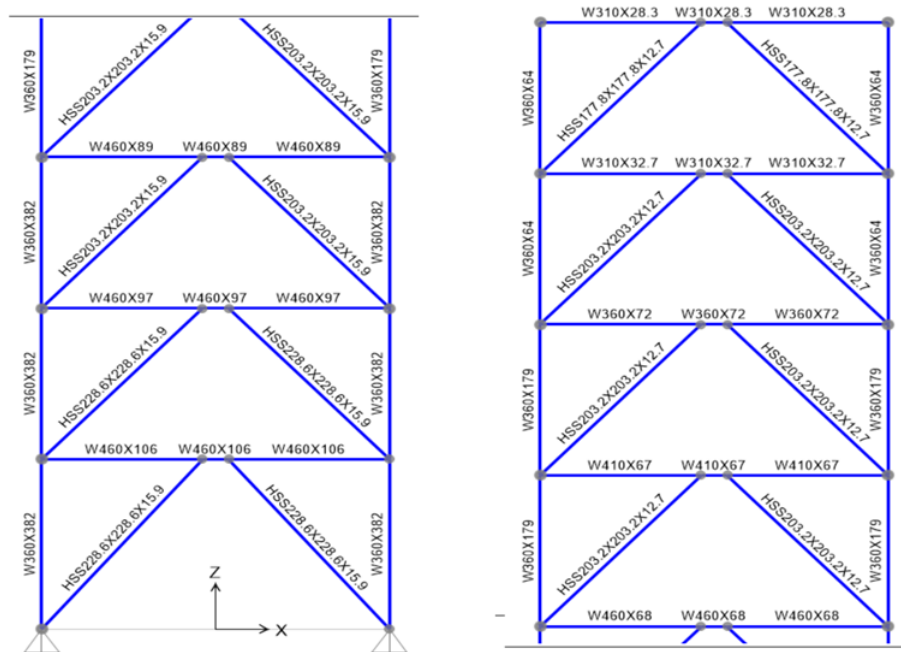


Figure 6-12 Sap2000 model for 8-storey structure soil class