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PERFORMANCE OF SEISMICALLY DEFICIENT EXISTING BRACED STEEL FRAME STRUCTURES WITH FLEXIBLE DIAPHRAGMS IN HALIFAX

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by

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Abstract

Concentrically braced frames (CBFs) have been one of the fundamental structural systems for lateral force resistance chosen by designers for low-rise steel construction since the early part of the twentieth century. CBFs designed using the building codes and standards of the 1960s were designed using the principle that they remained in the linearly elastic range. The current design philosophy of the 2010 National Building Code of Canada (NRCC 2010; CSA 2009) is based on the principles of capacity design and recognizes the cyclic inelastic behaviour of CBFs. Since no detailing or design requirements for an inelastic seismic response were included in structures designed with past building codes, these structures are likely to exhibit seismic deficiencies, including lack of lateral resistance and insufficient ductility. Guidelines for evaluating the performance of CBFs are required in order to provide recommendations for seismic evaluation and rehabilitation for such existing buildings, or future building codes.

The behaviour of one-storey steel structures built with the 1965 National Building Code of Canada (NRCC 1965) and CSA S16-65 (CSA 1965) under current building code standards for seismic design was studied in order to aid in establishing such guidelines. The response of a series of sixteen one-storey buildings with varying aspect ratios and heights was studied, subjected to ten synthetic and ten historical earthquake ground motions. The nonlinear seismic behaviour of the CBFs was determined using the analytical OpenSees, *Open System for Earthquake Engineering Simulation* (OpenSees 2011), model for nonlinear time-history dynamic analyses, including drift and ductility demands on the braces.

The intended performance levels under design earthquakes, as well as the acceptance criteria used in the braced frame analyses were established using FEMA P695 (FEMA 2009) criteria. In general, although acceptable performance was not achieved in all cases, the one-storey steel structures built with the 1965 National Building Code of Canada, on average, performed well, for the seven failure criteria outlined in this study.

Abstract

Concentrically braced frames (CBFs) have been one of the fundamental structural systems for lateral force resistance chosen by designers for low-rise steel construction since the early part of the twentieth century. CBFs designed using the building codes and standards of the 1960s were designed using the principle that they remained in the linearly elastic range. The current design philosophy of the 2010 National Building Code of Canada (NBCC) and CSA-S16-09 is based on the principles of capacity design and recognises the cyclic inelastic behaviour of CBFs. Since no detailing or design requirements for an inelastic seismic response were included in structures designed with past building codes, these structures are likely to exhibit seismic deficiencies, including lack of lateral resistance and insufficient ductility. Guidelines for evaluating the performance of CBFs are required in order to provide recommendations for seismic evaluation and rehabilitation for such existing buildings for future building codes.

The behaviour of one-storey steel structures built with the 1965 National Building Code of Canada (NRCC 1965) and CSA-S16-65 (CSA 1965) under current building code standards for seismic design was studied in order to aid in establishing such guidelines. The response of a series of sixteen one-storey buildings with varying aspect ratios and heights was studied, subjected to ten artificial and ten historical earthquake ground motions. The nonlinear seismic behaviour of the CBFs was determined using an analytical OpenSees, *Open System for Earthquake Engineering Simulation* (OpenSees 2011), model for nonlinear time history dynamic analysis, including drift and ductility demands on the braces.

The intended performance level in the design earthquakes, as well as the acceptance criteria used in the braced frame analysis were established using FEMA P695 (FEMA 2009) criteria. In general, although acceptable performance was not achieved in all cases, the one-storey steel structures built with the 1965 National Building Code of Canada, on average, performed well, for the seven failure criteria outlined in this study.

Résumé

Les cadres à contreventements concentriques (CCC) sont un des systèmes fondamentaux choisis par les concepteurs de bâtiments de faible hauteur en acier pour la résistance aux charges latérales depuis le début du vingtième siècle. Les CCC construit selon les codes de bâtiments et standards des années 1960 ont été conçus avec le principe qu'ils demeurent dans la zone d'élasticité linéaire. La philosophie de conception en vigueur dans le Code nationale de bâtiment 2010 et le CSA-S16-09 est basé sur le principe de calcul par capacité et constate le comportement inélastique cyclique des CCC. Les bâtiments conçus avec ces anciens codes n'avaient aucune exigence de détaillage ou conception particulier pour un comportement inélastique cyclique, donc il est probable que ces structures démontrent des déficiences sismiques, incluant un manque de résistance aux charges latérales et un manque de ductilité. Des principes pour l'évaluation de la performance des CCC sont requis pour fournir des recommandations pour l'évaluation sismique et des exigences pour la réhabilitation de ces structures existantes pour les futurs codes de bâtiment.

Le comportement de structures d'un étage conçus avec le Code nationale de bâtiment 1965 (NRCC 1965) et le CSA-S16-65 (CSA 1965) selon les normes sismiques des codes de bâtiment en vigueur a été étudier pour aider à établir ces recommandations. La performance d'une série de seize bâtiments d'un étage avec des rapports d'aspect et hauteurs variés a été étudiée, subie à dix enregistrements sismiques artificiels et historiques. Le comportement non-linéaire sismique des CCC a été déterminé avec un modèle analytique OpenSees, *Open System for Earthquake Engineering Simulation* (OpenSees 2011), pour des analyses non-linéaires dynamiques temporelles, incluant les demandes de déplacement et ductilité des contreventements.

Le niveau de performance visé avec les séismes de conception, et les critères d'acceptation dans l'analyse des cades contreventer a été établie avec FEMA P695 (FEMA 2009). Selon cette étude, en générale, même si les critères d'acceptation n'ont pas été atteints dans tous les cas, les bâtiments d'un étage conçu avec le Code nationale de bâtiment 1965 ont, en moyenne, performer bien pour les sept critères d'évaluation soulignée dans cette étude.

Acknowledgements

Firstly, I would like to thank Professor Colin A. Rogers for all of his support and guidance throughout the process of the completion of this research project.

I would also like to acknowledge all of the help that I received from fellow student Anthony Caruso-Juliano, and thank doctoral student Tom Morrison for all his assistance.

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None of this would have been possible without the unwavering support of all the people in my life. Thank you.

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Chapter 1 – Introduction

1.1 Background

Concentrically braced frames (CBFs) are defined in the CSA S16-2009 *Design of Steel Structures* Standard as "those in which the centre-lines of diagonal braces, beams and columns are approximately concurrent with little or no joint eccentricity" (CSA S16 2009). CBFs have been one of the fundamental structural systems for lateral force resistance chosen by designers for low-rise steel construction since the early part of the twentieth century. They were often combined with the use of infilled masonry walls or moment resisting frames to resist principally wind loads. Typical configuration of braced frames employed tension-only braces in either an X, as seen in Figure 1, or knee brace construction. CBF buildings also include the use of a horizontal lateral force resisting system consisting of a steel deck, considered as a flexible diaphragm, to transfer lateral loads to the CBFs. With the introduction of seismic requirements in the building codes of the 1960s they were often preferred over moment resisting frames as drift became an important design consideration (Bruneau et al. 1998).

Figure 1- CBF building under construction.

CBFs employ high elastic stiffness and resist lateral load based on axial action, with very little bending or flexural action. The initial design philosophy of codes of the 1960s for wind loads was that braces remained in the linearly elastic range. This principle was also

applied for seismic design initially and buildings designed using earlier codes were not detailed for inelastic seismic response. Later codes recognised the cyclic inelastic behaviour of CBFs and lead to the introduction of the capacity design procedures and detailing that are the basis of modern steel design (Bruneau et al. 1998).

The current 2010 National Building Code of Canada (NBCC) and CSA-S16-09 (CSA 2009) design philosophy is based on the principles of capacity design. This is where a specific element in the lateral force resisting system is designed to dissipate energy through inelastic response under seismic loads. The other elements of the frame are designed such that they are strong enough, while remaining elastic, so that the fuse element can dissipate the required energy. This ensures a proper hierarchy for yielding such that inelastic response is constrained to the specified element. In CBFs the element designed to dissipate energy is the brace itself, which requires a certain level of ductility to withstand the large deformations under the design earthquake. The NBCC prescribes that the building be able to undergo such deformations while remaining intact to enable evacuation, preventing major failure and loss of life.

This research will focus on the performance of CBFs designed using the codes of the 1960s where braces were designed using the principle that they remained in the linearly elastic range, using the current design philosophy of the 2010 National Building Code of Canada (NBCC) and CSA-S16-09.

1.2 Goal of Research

This research was conducted in order to aid in establishing guidelines for evaluating the performance of CBFs in one-storey steel structures built with the 1965 National Building Code of Canada (NBCC) and CSA-S16-65 (CSA 1965) under current building code standards for seismic design. These past codes incorporated no detailing or design requirements for an inelastic seismic response and as such these structures are likely to exhibit seismic deficiencies, including lack of lateral resistance and insufficient ductility.

The nonlinear seismic behaviour of the CBFs subjected to time history analysis is studied, including the drift and ductility demands on the braces. Such research is required in order applied for seismic design initially and buildings designed using earlier codes were not detailed for inelastic seismic response. Later codes recognised the cyclic inelastic behaviour of CBFs and lead to the introduction of the capacity design procedures and detailing that are the basis of modern steel design (Bruneau et al. 1998).

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1.3 Scope and Limitations

The response of a series of sixteen one-storey buildings with varying aspect ratios and heights was studied, subjected to ten artificial and ten historical earthquake ground motions. These structures were dimensioned and designed according to the 1965 National Building Code of Canada and CSA S16-1965 *Steel Structures for Buildings* for the city of Halifax. No ductility verifications were performed in the member selection and design, as was consistent with steel design practise in the 1960s. The lateral force resisting system consisted of CBFs along exterior walls with single angle bracing and considered the use of a horizontal lateral force resisting system consisting of a steel deck as a flexible diaphragm, to transfer lateral loads to the CBFs. The gravity load carrying system considered in the 1965 design consisted of open web steel joists, supported on cantilevered Gerber beams for the interior spans, with simply supported perimeter beams.

The study is limited to structures on soils of class C, as defined in the current NBCC 2010. Buildings considered were rectangular or square and symmetrical in plan with uniform mass, stiffness, and strength. As such, in-plane torsional effects were not considered in the design and analyses. The effect of non-structural components, including cladding and interior partitions, on the dynamic response of the buildings being studied was not considered in this report.

This report is complimentary to a study of similar scope by Caruso-Juliano (2012) evaluating the performance of CBFs in one-storey steel structures built with the 1965 National Building Code of Canada (NBCC) and CSA-S16-65 (CSA 1965) for the cities of Abbotsford and Montreal.

1.4 Overview of Report

This project report will begin with a literature review of the state of seismic design as prescribed by the current National Building Code of Canada as well as a brief review of past design practises and the history of Canadian codes and standards for seismic design. It will then describe the design procedure used to establish the braced bay geometry used

in this study according to the CSA S16-1965 *Steel Structures for Buildings* Standard and the 1965 National Building Code of Canada, as well as outline the different approach had the design been done with the current CSA S16-2009 *Design of Steel Structures* Standard and the 2010 National Building Code of Canada. The selection of earthquake time-history records will also be presented, as well as a brief overview of the development of the analytical OpenSees, *Open System for Earthquake Engineering Simulation* (OpenSees 2011), model for nonlinear time history dynamic analysis, accounting for the roof diaphragm flexibility and hysteretic behaviour of the braces. The intended performance level in the design earthquakes will be discussed, as well as the acceptance criteria used in the braced frame analysis. The results of the nonlinear analysis will then be presented and discussed. The report will also include an analysis of an example building using a 3D modelling program with dynamic modal analysis in order to compare the OpenSees analysis to the verification approach that would be taken by a design engineer according to the current NBCC 2010. The project report will end with conclusions on the performance of CBFs in one-storey steel structures built with the 1965 National Building Code of Canada under current building code standards for the city of Halifax.

Chapter 2 – Literature Review

2.1 History of Seismic Design in the NBCC

The National Building Code of Canada has evolved over the last 70 years as new research has emerged and as design philosophy has changed. This is especially evident in the area of seismic design and in the evaluation of seismic hazard. As such, the evaluation of seismic design force levels (represented by base shear) has differed with each publication of the NBCC.

The first seismic design provisions were based on the 1935 Uniform Building Code (UBC 1935) and published in the 1941 National Building Code of Canada (NRCC 1941). The design lateral force was based on the weight of the building, W, and a seismic force coefficient C, which varied from 0.02 to 0.05, to account for the bearing capacity of the soil (NRCC 1941; Mitchell et al. 2010). Base shear, V, was defined as:

$$V=CW \tag{2-1}$$

The 1953 National Building Code of Canada (NRCC 1953) saw the introduction of a seismic zoning map, as presented in Figure 2, with four zones of varying seismic hazard, based on historical earthquake data. Zone 0 did not require any specific seismic design, zone 1 represented the base case for design lateral load, zone 2 required a multiplier of 2 on the design lateral load, and zone 3 required a multiplier of 4 on the design lateral load. The seismic force coefficient C_i was also modified to be a function of the building's stiffness, based on its number of storeys above the "level i" under consideration. The seismic weight, W_i, was defined as the dead load plus 25% of the design snow load for "level i" under consideration (NRCC 1953; Mitchell et al. 2010). Force at "level i", F_i, was defined as:

$$F_i=C_iW_i \tag{2-2}$$

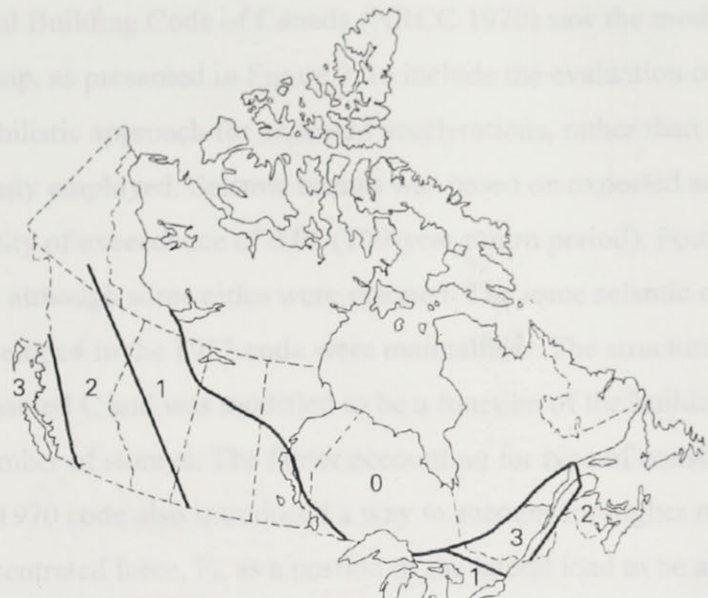


Figure 2 - Seismic zoning map from the 1953 National building code of Canada (NRCC 1953).

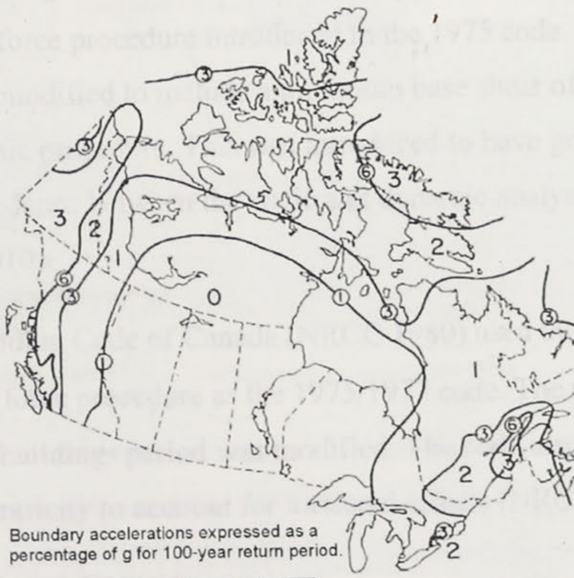
In the 1960 National Building Code of Canada (NRCC 1960) seismic design requirements were the same as the 1953 code; however, it introduced the need to account for torsional effects without specifying a precise procedure (NRCC 1960; Mitchell et al. 2010).

The 1965 National Building Code of Canada (NRCC 1965) used the same seismic zoning map introduced in the 1953 code with the same multipliers on the design lateral load; however it redefined them as the seismic regionalization factor, R. It also introduced a factor to account for the type of lateral force resisting system, C, an importance factor, I, to account for the intended use of the building, and a foundation factor, F, to account for site soil conditions. The coefficient representing the building's stiffness, based on the number of storeys, was modified and redefined as the structural flexibility factor, S. The design lateral load was specified as being linearly distributed to respective floor levels based on height and weight. The code also introduced guidelines on accounting for torsional effects (NRCC 1965; Mitchell et al. 2010). Base shear, V, was defined as:

$$V=RCIFSW \qquad [2\text{-}3]$$

The 1970 National Building Code of Canada (NRCC 1970) saw the modification of the seismic zoning map, as presented in Figure 3, to include the evaluation of seismic hazard based on a probabilistic approach for expected accelerations, rather than the historic approach previously employed. Seismic hazard was based on expected accelerations having a probability of exceedance of 0.01 (100-year return period). Four seismic zones were maintained, although some cities were rezoned. The same seismic regionalization factors, R, as developed in the 1953 code were maintained. The structural flexibility factor, S, was renamed C and was modified to be a function of the building's period rather than its number of storeys. The factor accounting for type of construction, C, was renamed K. The 1970 code also introduced a way to account for higher mode effects by specifying a concentrated force, F_t, as a portion of the lateral load to be applied at the top of the building and a reduced overturning moment (NRCC 1970; Mitchell et al. 2010). Base shear, V, was defined as:

$$V = 1/4\, R(KCIFW) \qquad [2\text{-}4]$$



Zone	3	2	1	0
R	4	2	1	0

Figure 3 – Seismic zoning map from the 1970 National building code of Canada (NRCC 1970).

The 1975 National Building Code of Canada (NRCC 1975) used the same seismic zoning map introduced in the 1970 code; however the seismic regionalization factor, R was eliminated. Instead the horizontal design ground acceleration, A, was introduced for

zones 0 through 3, and a seismic response factor, S, was introduced based on the buildings period. The factors accounting for type of construction, K were modified from the 1970 code to account for the effects of damping, ductility and energy absorption, and a new foundation factor, F, to account for intermediate site soil conditions was introduced. The code also modified the guidelines on accounting for torsional effects that had been introduced in the 1965 code. Dynamic analysis, as an alternative method for determining lateral loads, was also introduced in the 1975 code based on a scaled response spectrum with 5% damping. Scaling was specified according to the horizontal design ground acceleration for zones 0 through 3. A structural ductility factor, μ, was specified for calibrating the response spectrum. The response spectra of buildings with shorter periods were divided by $\sqrt{2\mu - 1}$, whereas those with longer periods were divided by μ (NRCC 1975; Mitchell et al. 2010). Base shear, V, was defined as:

$$V=ASKIFW \qquad [2\text{-}5]$$

The 1977 National Building Code of Canada (NRCC 1977) used the same seismic zoning map and design lateral force procedure introduced in the 1975 code. The provisions for dynamic analysis were modified to include a minimum base shear of 90% of the value calculated using the static procedure. This was introduced to have greater compatibility of the probability of exceedance between the static and dynamic analysis methods (NRCC 1977; Mitchell et al. 2010).

The 1980 National Building Code of Canada (NRCC 1980) used the same seismic zoning map and design lateral force procedure as the 1975/1977 code. The seismic response factor, S, based on the buildings period was modified. The code also included guidelines on calculating the eccentricity to account for torsional effects (NRCC 1980; Mitchell et al. 2010).

The 1985 National Building Code of Canada (NRCC 1985) saw the modification of the seismic zoning map, as presented in Figure 4, to include the evaluation of seismic hazard based on a new point source model with a probability of exceedance of 10% in 50 years (return period of 475 years). As well the number of seismic zones was increased from four to seven, each being assigned a velocity zonal ratio, v. The seismic response factor, zones 0 through 3, and a seismic response factor, S, was introduced based on the buildings period. The factors accounting for type of construction, K were modified from the 1970 code to account for the effects of damping, ductility and energy absorption, and a new foundation factor, F, to account for intermediate site and soil conditions was introduced. The code also modified the guidelines on accounting for torsional effects that had been introduced in the 1965 code. Dynamic analysis, as an alternative method for determining lateral loads, was also introduced in the 1975 code based on a normal response spectrum with 5% damping. Scaling was specified according to the horizontal design ground acceleration for zones 0 through 3. A structural ductility factor, μ, was specified for calibrating the response spectrum. The response spectra of buildings with shorter periods were divided by $\sqrt{2\mu-1}$, whereas those with longer periods were divided by μ (NRCC 1975; Mitchell et al. 2010). Base shear, V, was defined as:

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The 1980 National Building Code of Canada (NRCC 1980) used the same seismic zoning map and design lateral force procedure as the 1975/1977 code. The seismic response factor, S, based on the buildings period was modified. The code also included guidelines on calculating the eccentricity to account for torsional effects (NRCC 1980; Mitchell et al. 2010).

The 1985 National Building Code of Canada (NRCC 1985) saw the modification of the seismic zoning map, as presented in Figure 4, to include the evaluation of seismic hazard based on a new point-source model with a probability of exceedance of 10% in 50 years (return period of 475 years). As well the number of seismic zones was increased from four to seven, each defined a velocity-zonal ratio, v. The seismic response factor, S, based on the buildings period was modified. The code allowed the building period to be determined by modal analysis, to a maximum of 1.2 times the specified empirical value. The provisions for dynamic analysis were modified to include a minimum base shear of 100% of the value calculated using the static procedure, as compared to the 90% previously allowed. The code also modified the guidelines on accounting for torsional effects such that accidental eccentricity was increased from 0.05D to 0.10D (NRCC 1985; Mitchell et al. 2010). Base shear, V, was defined as:

$$V = vSKIFW \qquad [2\text{-}6]$$

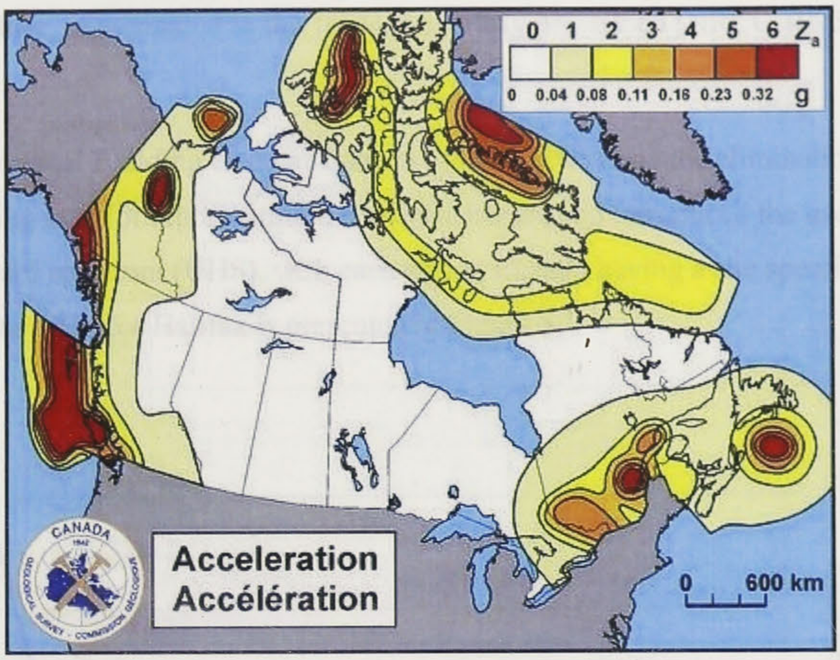

Figure 4 - Seismic zoning map from the 1985 National building code of Canada (NRCC 1985).

The 1990 National Building Code of Canada (NRCC 1990) used the same seismic zoning map as the 1985 code. The factor accounting for type of construction, K, was replaced by the force modification factor, R. The introduction of the force modification factor, R, was the NBCC's first recognition of a buildings capacity to dissipate energy during a seismic event through inelastic behaviour and it required specific design and detailing requirements be met according to the R-value chosen. A calibration factor, U, was introduced to ensure the base shear was compatible with previous editions of the NBCC. A new foundation factor, F, to account for very soft soil conditions was introduced. The 1990 code also included drift limits of $0.02h_s$ for normal buildings and $0.01h_s$ for post-

disaster buildings, where h_s is the height of the building (NRCC 1990; Mitchell et al. 2010). Base shear, V, was defined as:

$$V=U\,(vSIFW)/R \qquad [2\text{-}7]$$

The 1995 National Building Code of Canada (NRCC 1995) used the same seismic zoning map and design lateral force procedure as the 1990 code. Additional force modification factors, R, were introduced. The code also modified the expressions for building periods and guidelines on accounting for torsional effects. The provisions for dynamic analysis were modified to include a minimum base shear of 80% of the value calculated using the static procedure, as compared to the 100% previously prescribed (NRCC 1995; Mitchell et al. 2010).

The 2005 National Building Code of Canada (NRCC 2005) saw the elimination of the seismic zoning map for representing the seismic hazard and introduced the use of a uniform hazard spectrum (UHS), with each city in Canada having a site specific response spectrum. The UHS for Halifax is presented in Figure 5.

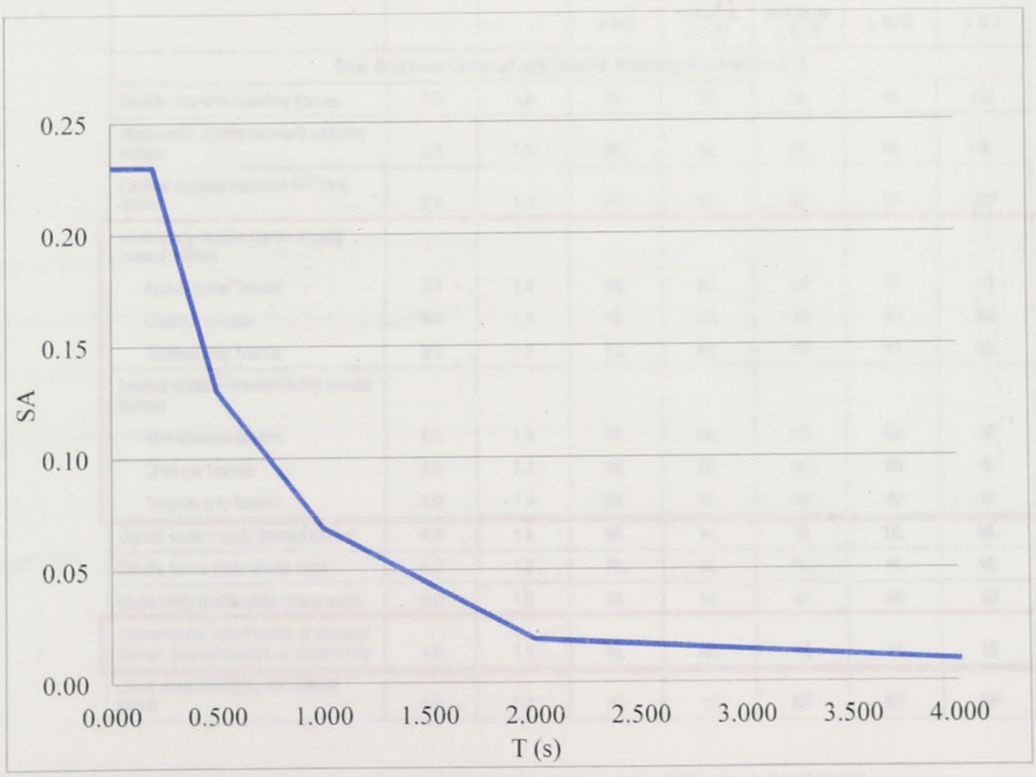

Figure 5 - Uniform hazard spectrum for Halifax from the 2005 National building code of Canada (NRCC 2005).

These UHS had a probability of exceedance of 2% in 50 years (2475-year return period), in comparison to the previous probability of exceedance of 10% in 50 years. Dynamic analysis was established as the preferred design method, and was imposed for irregular structures. Base shear calculation in the 2005 NBCC was based on the spectral acceleration at the fundamental building period, S (T_a). The higher mode participation factor was renamed M_v and the importance factor for earthquake load, I_E. The ductility-related force modification factor, R_d, was introduced to account for the buildings capacity to dissipate energy during a seismic event through inelastic behaviour. As well, the overstrength-related force modification factor, R_o, was introduced to account for reserved strength in a structure. For CBFs values of R_d and R_o were presented in Table 4.1.8.9 of the NBCC 2005, reproduced in Figure 6, for three types of CBFs: Moderately ductility (MD), limited ductility (LD), and conventional construction. Details for these types of CBFs are outlined in CSA-09 and presented in Section 2.3 of this report.

Type of SFRS	R_d	R_o	Restrictions[2]				
			Cases Where $I_EF_aS_a(0.2)$				Cases Where $I_EF_vS_a(1.0)$
			< 0.2	≥ 0.2 to < 0.35	≥ 0.35 to ≤ 0.75	> 0.75	> 0.3
Steel Structures Designed and Detailed According to CAN/CSA-S16							
Ductile moment-resisting frames	5.0	1.5	NL	NL	NL	NL	NL
Moderately ductile moment-resisting frames	3.5	1.5	NL	NL	NL	NL	NL
Limited ductility moment-resisting frames	2.0	1.3	NL	NL	60	30	30
Moderately ductile concentrically braced frames							
Non-chevron braces	3.0	1.3	NL	NL	40	40	40
Chevron braces	3.0	1.3	NL	NL	40	40	40
Tension only braces	3.0	1.3	NL	NL	20	20	20
Limited ductility concentrically braced frames							
Non-chevron braces	2.0	1.3	NL	NL	60	60	60
Chevron braces	2.0	1.3	NL	NL	60	60	60
Tension only braces	2.0	1.3	NL	NL	40	40	40
Ductile eccentrically braced frames	4.0	1.5	NL	NL	NL	NL	NL
Ductile frame plate shear walls	5.0	1.6	NL	NL	NL	NL	NL
Moderately ductile plate shear walls	2.0	1.5	NL	NL	60	60	60
Conventional construction of moment frames, braced frames or shear walls	1.5	1.3	NL	NL	15	15	15
Other steel SFRS(s) not defined above	1.0	1.0	15	15	NP	NP	NP

Figure 6 - Excerpt from Table 4.1.8.9 from the 2005 National building code of Canada (NRCC 2005).

The seismic weight, W, was modified to include the dead load (including partitions at 0.5 kPa), 25% of the design snow load, 60% of storage live loads, and the full contents of any tanks. The code also modified the guidelines on accounting for torsional effects and introduced a calculation for determining a building's torsional sensitivity based on the ratio between maximum and average storey displacements at extreme points of the structure. The 2005 National Building Code of Canada also introduced a defined set of structural irregularities to be avoided by designers, and established specific design requirements should a building have one of these irregularities. As well, it introduced height restrictions for the selection of a building's lateral-force resisting system. The provisions for dynamic analysis were modified to include a minimum base shear of 80% of the value calculated using the static procedure if the structure is regular, or 100% if the structure is irregular. The 2005 code also included drift limits of $0.025h_s$ for normal buildings, $0.020h_s$ for schools, and $0.01h_s$ for post-disaster buildings. Drift limits were specified as those resulting from elastic deflection i.e. multiplied by R_dR_o/I_E when drifts are determined based on the modified design spectrum (NRCC 2005; Mitchell et al. 2010). Base shear, V, was defined as:

$$V = S(T_a)M_vI_EW/R_dR_o \qquad [2\text{-}8]$$

2.2 Seismic Design Philosophy in 2010 NBCC

With the 2010 National Building Code of Canada (NRCC 2010), the uniform hazard spectrums were modified for the eastern cities, including Halifax as per Figure 7, while western cities maintained the same UHS as defined in the 2005 code. The UHS design spectrum was based on ordinates at periods of 0.2, 0.5, 1.0, and 2.0 s with a probability of exceedance of 2% in 50 years as seen in the 2005 NBCC.

As well, new lateral force resisting systems were added to the code with additional force modification factors R_d and R_o (NRCC 2010; Mitchell et al. 2010).

The seismic weight, W, was modified to include the dead load (including partitions at 0.5 kPa), 25% of the design snow load, 60% of storage live loads, and the full contents of any tanks. The code also modified the guidelines on accounting for torsional effects and introduced a calculation for determining a building's torsional sensitivity based on the ratio between maximum and average storey displacements at extreme points of the structure. The 2005 National Building Code of Canada also introduced a defined set of structural irregularities to be avoided by designers, and established specific design requirements should a building have one of these irregularities. As well, it introduced height restrictions for the selection of a building's lateral-force resisting system. The provisions for dynamic analysis were modified to include a minimum base shear of 80% of the value calculated using the static procedure if the structure is regular, or 100% if the structure is irregular. The 2005 code also included drift limits of 0.025h, for normal buildings, 0.020h, for schools, and 0.01h, for post-disaster buildings. Drift limits were specified as those resulting from elastic deflection i.e. multiplied by R_dR_o/I_E when drifts are determined based on the unmodified design spectrum (NRCC 2005; Mitchell et al. 2010). Base shear, V, was defined as:

$$V=S(T_a)M_vI_EW/R_dR_o \qquad [2\text{-}8]$$

2.2 Seismic Design Philosophy in 2010 NBCC

With the 2010 National Building Code of Canada (NRCC 2010), the uniform hazard spectrums were modified for the eastern cities, including Halifax as per Figure 7, while western cities maintained the same UHS as defined in the 2005 code. The UHS design spectrum was based on ordinates at periods of 0.2, 0.5, 1.0, and 2.0 s with a probability of exceedance of 2% in 50 years as seen in the 2005 NBCC.

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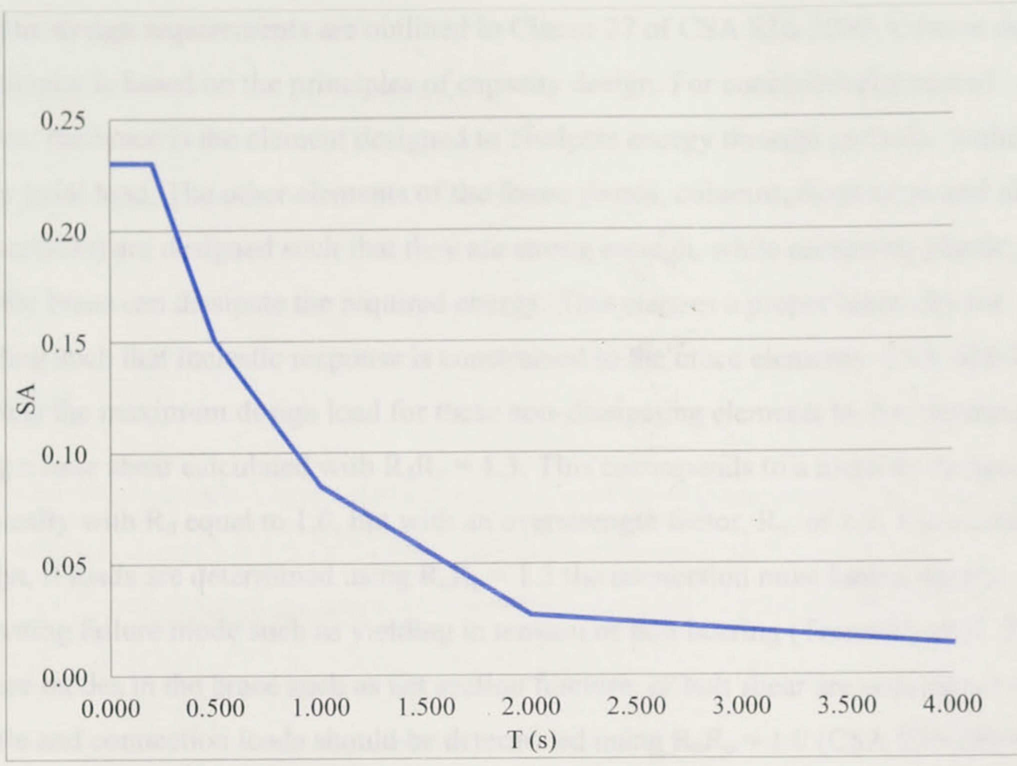

Figure 7 - Uniform hazard spectrum for Halifax from the 2010 National building code of Canada (NRCC 2010).

The value of base shear required for design by the 2010 NBCC varies according to the seismic force factors R_dR_o, which are assigned based on the level of ductility and overstrength that can be attributed to the structural system. This level of ductility is based on the system's ability to dissipate energy through inelastic deformations, and the minimum overstrength of the particular system. Each category of ductility has matching detailing requirements to ensure the anticipated inelastic behaviour is attainable. These requirements are outlined in CSA S16 for steel structures and CSA A23.3 for concrete structures. If inelastic deformations are not possible without a loss of resistance the system should be designed elastically with R_d equal to 1.0. For concentrically braced frames, systems are classified as "conventional construction", "limited ductility", or "moderate ductility" (NRCC 2010; Mitchell et al. 2010).

2.3 Concentrically Braced Frames in S16-09

Current design philosophy in CSA S16-2009 *Design of Steel Structures* Standard is based on limits state design, using load factors and material resistance factors.

Seismic design requirements are outlined in Clause 27 of CSA S16-2009. Current design philosophy is based on the principles of capacity design. For concentrically braced frames, the brace is the element designed to dissipate energy through inelastic straining under axial load. The other elements of the frame (beam, columns, diaphragm and all connections) are designed such that they are strong enough, while remaining elastic, so that the brace can dissipate the required energy. This ensures a proper hierarchy for yielding such that inelastic response is constrained to the brace elements. CSA S16-2009 restricts the maximum design load for these non-dissipating elements to that determined using a base shear calculated with $R_dR_o = 1.3$. This corresponds to a member designed elastically with R_d equal to 1.0, but with an overstrength factor, R_o, of 1.3. For connection design, if loads are determined using $R_dR_o = 1.3$ the connection must have a ductile governing failure mode such as yielding in tension or bolt bearing (Tremblay et al. 2009). Failure modes in the brace such as net section fracture, or bolt shear are considered non ductile and connection loads should be determined using $R_dR_o = 1.0$ (CSA S16 2009).

CSA S16-2009 specifies certain material and connection requirements in order to promote adequate post-yield behaviour. This includes that the specified minimum yield stress, F_y, of steel not exceed 350 MPa for these elements. It also specifies the use of probable yield stress, R_yF_y, not less than 460 MPa for HSS section and 385 MPa for other sections with R_y equal to 1.1. This ensures a proper hierarchy for yielding where the seismic load is based on the actual (probable) yield stress of the brace (CSA S16 2009).

In the CSA S16-2009 Standard Clause 27.1.9 was introduced which established specific protected zones where no discontinuity or any stress concentrations are allowed to be introduced through welding, cutting, or penetration. For the concentrically braced frame this is the brace element itself over its entire length as well as the connections to beams and columns including gusset plates. These are designated as protected zones since they are the regions where large inelastic strains are expected to occur. As well, inelastic rotational demands are placed on the connections. Any discontinuity in these elements can lead to premature fracture under cyclic inelastic response (CSA S16 2009).

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2.3.1 Moderate Ductility CBFs

Seismic design requirements for moderately ductile (MD) concentrically braced frames are outlined in Clause 27.5 of CSA S16-2009. This system is defined with an R_d of 3.0, and a R_o of 1.3. Tension-compression, tension-only, and chevron MD brace systems may be used, however, knee bracing and K-bracing are not allowed according to CSA S16-2009 due to the possibility of creating a plastic hinge in the column if one brace was to buckle. The specific requirements for these braces are outlined in Clauses 27.5.2 (CSA S16 2009).

CSA S16-2009 also establishes certain restrictions regarding the brace slenderness ratio, kL/r, which is limited to a maximum of 200. As the brace slenderness ratio, kL/r, is decreased the capacity of the brace to dissipate energy under cyclic inelastic loading is increased (CSA S16 2009, Tremblay et al. 2003).

As well, Clause 27.5.3.2 establishes specific width-to-thickness ratios in order to protect against cyclic local buckling which can lead to early fracture (CSA S16 2009).

In the MD system the principles of capacity design are introduced through the use of probable brace resistance to design the "protected elements". The probable tension resistance, probable compression resistance, and probable post-buckling compressive resistance of the brace are taken respectively as:

$$T_u = A_g R_y F_y \quad [2\text{-}9]$$

$$C_u = \min\begin{Bmatrix} A_g R_y F_y \\ 1.2 C_r/\phi \end{Bmatrix} \quad [2\text{-}10]$$

$$C'_u = \min\begin{Bmatrix} 0.2 A_g R_y F_y \\ C_r/\phi \end{Bmatrix} \quad [2\text{-}11]$$

In calculating the probable compression resistance and the probable post-buckling compressive resistance of the brace, C_r is calculated using $R_y F_y$. CSA S16-2009 prescribes the use of two loading conditions. The first is the compression braces' reaching their probable compression resistance and the second is the compression braces' reaching their probable post-buckling compressive resistance. Each of these cases is to be

combined with the tension braces' reaching their probable tension resistance (CSA S16 2009).

The non-dissipating elements of the frame (beam, columns, diaphragm and all connections) are designed for the worst case of gravity load and brace forces equal to the onset of simultaneous yielding in the tension and compression braces (either probable compression resistance or probable post-buckling compressive whichever produces the worst effect). CSA S16-2009 restricts the maximum design load for these non-dissipating elements to that determined using a storey shear based on $R_dR_o = 1.3$ (CSA S16 2009).

2.3.2 Limited Ductility CBFs

Seismic design requirements for limited ductile (LD) concentrically braced frames are outlined in Clause 27.6 of CSA S16-2009. This system is defined with an R_d of 2.0, and a R_o of 1.3. Detailing requirements are less stringent as compared to MD systems, with a reduced ability to dissipate energy through inelastic deformations. As in MD bracing systems tension-compression, tension-only, and chevron brace systems may be used, as well as knee bracing and K-bracing where the columns are designed for the induced bending and struts are provided. The specific requirements for these braces are outlined in Clause 27.6.2 (CSA S16 2009).

For LD braces, the restrictions regarding the brace slenderness ratio, kL/r, are less severe than for the MD system and slenderness is limited to a maximum of 300. The bracing width-to-thickness ratio requirements are also less severe as defined in Clause 27.6.3.2 since braces with kL/r greater than 200 are expected to have very limited inelastic compressive strains and there is less risk of cyclic local buckling (CSA S16 2009).

2.3.3 Conventional Construction CBFs

Seismic design requirements for conventional construction are outlined in Clause 27.11 of CSA S16-2009. This system is defined with an R_d of 1.5, and a R_o of 1.3. The conventional construction system incorporates the principles of capacity design indirectly without particular detailing requirements. It is based on the concept that inherent in current design and construction practise there is some capacity of the brace to dissipate energy through localized yielding and friction. Clause 27.11.1 specifies that for

$I_E F_a S_a(0.2)$ greater than 0.45, diaphragms and connections of primary framing members be designed such that the failure mode is ductile, or for gravity loads combined with a seismic load multiplied by R_d =1.50. These minimum requirements were established to avoid brittle failure in connections and promote energy dissipation in the brace (CSA S16 2009).

2.3.4 Inelastic Response of CBFs

CBF performance is governed by its cyclic axial load response. Initially under compressive load the brace behaves elastically and deforms axially until buckling, when the brace deflects transversely. This deflection causes the creation of moments in the brace, which when they equal the plastic moment of the braces create a plastic hinge. Under increased axial load and axial deformation the brace continues to deflect transversely which causes a decrease in axial resistance of the brace due to flexure-axial load interaction (moment at the midlength of the brace cannot surpass the plastic moment). The brace retains a residual axial and transverse deflection when unloaded. During the tension cycle which follows the brace behaves elastically and deforms axially. When reloaded in compression the residual transverse deflection causes a reduction in buckling capacity and the length of the elastic buckling plateau is reduced with each subsequent cycle. This hysteretic behaviour is highly dependent on the slenderness ratio of the braces (Bruneau et al. 1998). Brace buckled shapes are shown in Figure 8 from cyclic testing by Morrison (2012).

The quantification of a system's capacity for energy dissipation is measured by the area enclosed by its force vs. deformation hysteresis curve (see Figure 9). Braces with lower slenderness ratios have larger hysteresis curve areas, and therefore dissipate more energy than slender braces. As well braces with lower slenderness ratios have compression capacities which approach the tension capacity (Bruneau et al. 1998).

Figure 8 - Various buckled shapes of cross braces during testing by Morrison (2012).

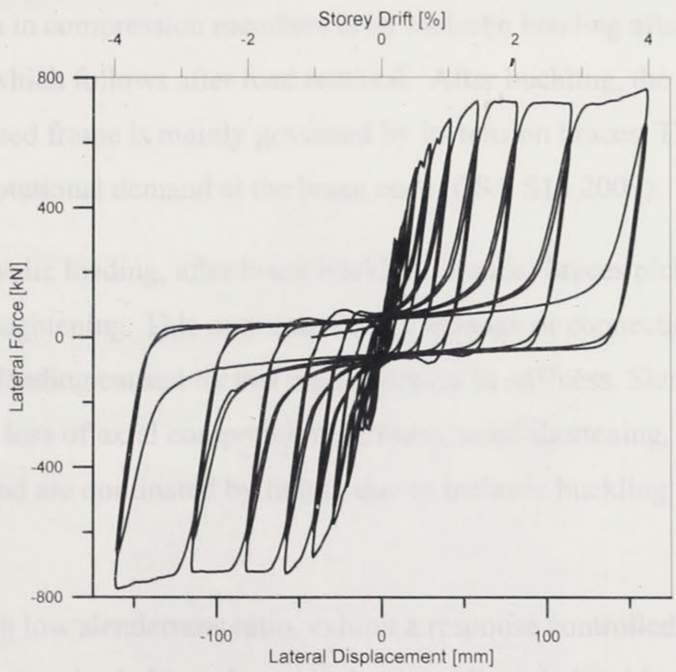

Figure 9 - Lateral force vs. displacement hysteretic performance of plain cross brace sample by Morrison (2012).

In tension-only systems energy dissipation is achieved by brace yielding and elongation. These systems are typically designed with braces having higher slenderness ratios than tension-compression systems, especially in older building where the slenderness ratio

often exceeded 300. The lateral stiffness of the frame is therefore governed mainly by the tension brace as the compression brace buckles under low axial load due to a higher slenderness ratio. As such, in the buckled configuration, these frames have little stiffness. Under cyclic loading the braces accumulate residual axial displacements as seen in the compression-tension systems. Frame stiffness is lost when there is zero frame displacement (Bruneau et al. 1998).

Brace ductility demand is measured by the ratio of maximum total deformation to the deformation at yield:

$$\mu = \delta/\delta_{yield} \qquad [2\text{-}12]$$

According to Tremblay (2002), the ductility demand ratio for CBFs with tension-compression braces is around 2 or 3.

2.3.5 Failure Modes of CBFs

Energy dissipation in compression members is by inelastic bending after buckling, and in the straightening which follows after load reversal. After buckling, the capacity of a concentrically braced frame is mainly governed by its tension braces. The buckling of braces induces a rotational demand at the brace ends (CSA S16 2009).

During reversed cyclic loading, after brace buckling, slender braces pick up axial load quickly during straightening. This may cause brace damage or connection failure, due to an impact type of loading caused by this rapid increase in stiffness. Slender braces are characterized by a loss of axial compression stiffness, axial shortening, and loss of tangent stiffness and are dominated by failure due to inelastic buckling (Bruneau et al. 1998).

Stocky braces, with low slenderness ratio, exhibit a response controlled by yielding and local buckling. At the plastic hinge located a brace midlength, local buckling causes a loss of moment capacity and a loss in axial capacity. This leads to a reduced energy dissipation capacity (Bruneau et al. 1998).

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Observed modes of failure in CBFs include, failure of the gross section of the brace, fracture of the net cross section of the brace (at the connection), or connection failure at the bolts or welds (Tremblay et al. 1996).

2.3.6 Connections

Connections in CBFs consist of gusset plates either welded or bolted to the frame's columns and beam. They are designed such that they are strong enough, while remaining elastic, that the brace can dissipate the required energy as per CSA-S16-09. Specific rules are outlined in CSA S16-09 for the case where the brace buckles in-the-plane or out-of-plane of the gusset. For in-the-plane buckling the connection must have a flexural strength equal to or greater than the nominal in-plane bending strength of the brace to prevent hinges forming in the connection. For out-of-plane buckling the connection the gusset must be detailed to permit a hinge line to form in the gusset. This is accomplished by leaving a distance equal to twice the gusset plate thickness between the brace and the nominal line of unrestrained bending as seen in Figure 10 (Bruneau et al. 1998, CSA-S16-09).

Figure 10 - Brace connection designed according to CSA S16-2009 Standard.

Non ductile limit states for brace connection failure include tension failure at the brace net section, and bolt shear failure. It has also been demonstrated that bolting geometry may affect ductility, as connections with larger bolt spacings have increased ductility (Castonguay and Tremblay 2010).

Connection design in CSA S16-65 was not based on the incorporation of any design or detailing requirement as prescribed by the capacity design principles of the current codes. The tension forces resulting from the static analysis of the braced frame would have been directly applied using a free body diagram for connection design. As well, it should be noted that connection design was based on an allowable stress approach.

2.4 Calculation of the Fundamental Period

In the 2010 NBCC the value of base shear required for design is related to the building's period of vibration for the static analysis case. If dynamic analysis is performed the base shear is still limited to a fundamental period calculated based on an empirical formula as a function of the building's height, h_n. For CBFs:

$$T_a \cong 0.025h_n \text{ or } T \cong 0.050h_n \text{ if verified by dynamic analysis} \qquad [2\text{-}13]$$

Tremblay et al. (1996) showed that for one-storey buildings with flexible diaphragms the use of the NBCC period resulted in conservative designs and that the in-plane flexibility of the diaphragm should be accounted for in determining the building period. This was also shown in Tremblay & Rogers (2011) where it was demonstrated that the majority of one-storey structures had periods longer than the limit prescribed by the code of $2T_a \cong 0.050h_n$ if verified by dynamic analysis. As well, Lamarche et al. (2009) pointed out that use of the NBCC empirical equation would result in one period of vibration for a given building, where in fact, the stiffness of the structure would most likely vary in the two principal directions. Lamarche et al. (2009) also concluded that most single-storey braced frames had fundamental periods in between the limits of $0.025h_n$ and $0.050h_n$, and that the relationship between the spectral acceleration and these two limits is not linear.

Medhekar proposed a methodology based on the Rayleigh method, accounting for the influence of brace and diaphragm stiffness (Tremblay & Rogers 2011):

$$T \cong 2\pi\sqrt{\frac{(K_B+K_D)W}{gK_BK_D}} \quad [2\text{-}14]$$

Where:

$$\Delta_B = \frac{PL}{AE}cos\theta \quad \text{and } K_B = \frac{1}{\Delta_B} \quad [2\text{-}15]$$

$$\Delta_D = \Delta_F + \Delta_S = \frac{5wL^4}{384EI} + \frac{wL^2}{8G'b} \quad \text{and } K_D = \frac{1}{\Delta_D} \quad [2\text{-}16]$$

The stiffness of the vertical bracing, K_B, is based on the inverse of the lateral deformation of the vertical bracing, Δ_B. The in-plane stiffness of the diaphragm, K_D, is based on the inverse of the diaphragm deflection at midspan calculated by combing the shear and flexural in-plane deformations, $\Delta_D = \Delta_S + \Delta_F$. The diaphragm deflection is typically governed by the shear, rather than flexural deformations (Lamarche et al. 2009).

Lamarche et al. (2009) showed that flexible diaphragms have considerable effect on the dynamic response of single-storey braced frames. As well, they demonstrated the impact of non-structural components on the dynamic response of single-storey braced frames under low amplitude excitation.

2.5 Summary

Current seismic design philosophy has evolved since its first introduction into the NBCC to include design and detailing requirements which account for the inelastic response of the braces in the CBF system. It remains unclear, however, how CBFs designed within the framework of earlier codes perform, when subjected to evaluation according to current seismic design codes and standards.

Analysis of CBFs designed with codes where the braces were designed using the principle that they remained in the linearly elastic range, such as the 1965 National Building Code of Canada (NBCC) and CSA-S16-65 is required in order to provide recommendations for seismic evaluation and rehabilitation requirements for such existing buildings for future building codes.

Chapter 3 – Design Review of Single-Storey Steel Buildings in Canada

3.1 Overview

Dimensioning of the structural elements for the buildings studied in the braced bays was based on the static calculations procedure. Design of a typical one-storey building was carried out according to the 1965 National Building Code of Canada (NRCC 1965) and the CSA S16-1965 *Steel Structures for Buildings* Standard (CSA 1965) and using normal design procedures as would be common for a steel designer at the time. Selection tables as presented in the *Handbook of Steel Construction* first edition 1965 were used for selection of optimum beam and column sections. These selection tables were based on the design provisions of CSA S16-1965.

As presented in Table 1, a series of buildings with varying aspect ratios and heights was studied in order to account for a diverse period range and behaviour of one-storey braced frame buildings.

Table 1 - Dimensions of buildings analyzed in study.

Area (m^2)		Aspect Ratio		L (m)	W (m)	# bays L	# bays W	Bay - L (m)	Bay - W (m)	H (m)
A	600	1H	1	24.5	24.5	4	4	6.124	6.124	4.00
		2H	1.5	30.0	20.0	6	4	5.000	5.000	5.000
		3H	2	34.6	17.3	6	3	5.774	5.774	6.000
		4H	2.5	38.7	15.5	5	2	7.746	7.746	7.000
B	1800	5H	1	42.4	42.4	6	6	7.071	7.071	5.000
		6H	1.5	52.0	34.6	6	4	8.660	8.660	6.000
		7H	2	60.0	30.0	8	4	7.500	7.500	7.000
		8H	2.5	67.1	26.8	10	4	6.708	6.708	8.000
C	3000	9H	1	54.8	54.8	7	7	7.825	7.825	6.000
		10H	1.5	67.1	44.7	9	6	7.454	7.454	7.000
		11H	2	77.5	38.7	10	5	7.746	7.746	8.000
		12H	2.5	86.6	34.6	10	4	8.660	8.660	9.000
D	4200	13H	1	64.8	64.8	8	8	8.101	8.101	7.000
		14H	1.5	79.4	52.9	9	6	8.819	8.819	8.000
		15H	2	91.7	45.8	10	5	9.165	9.165	9.000
		16H	2.5	102.5	41.0	10	4	10.247	10.247	10.000

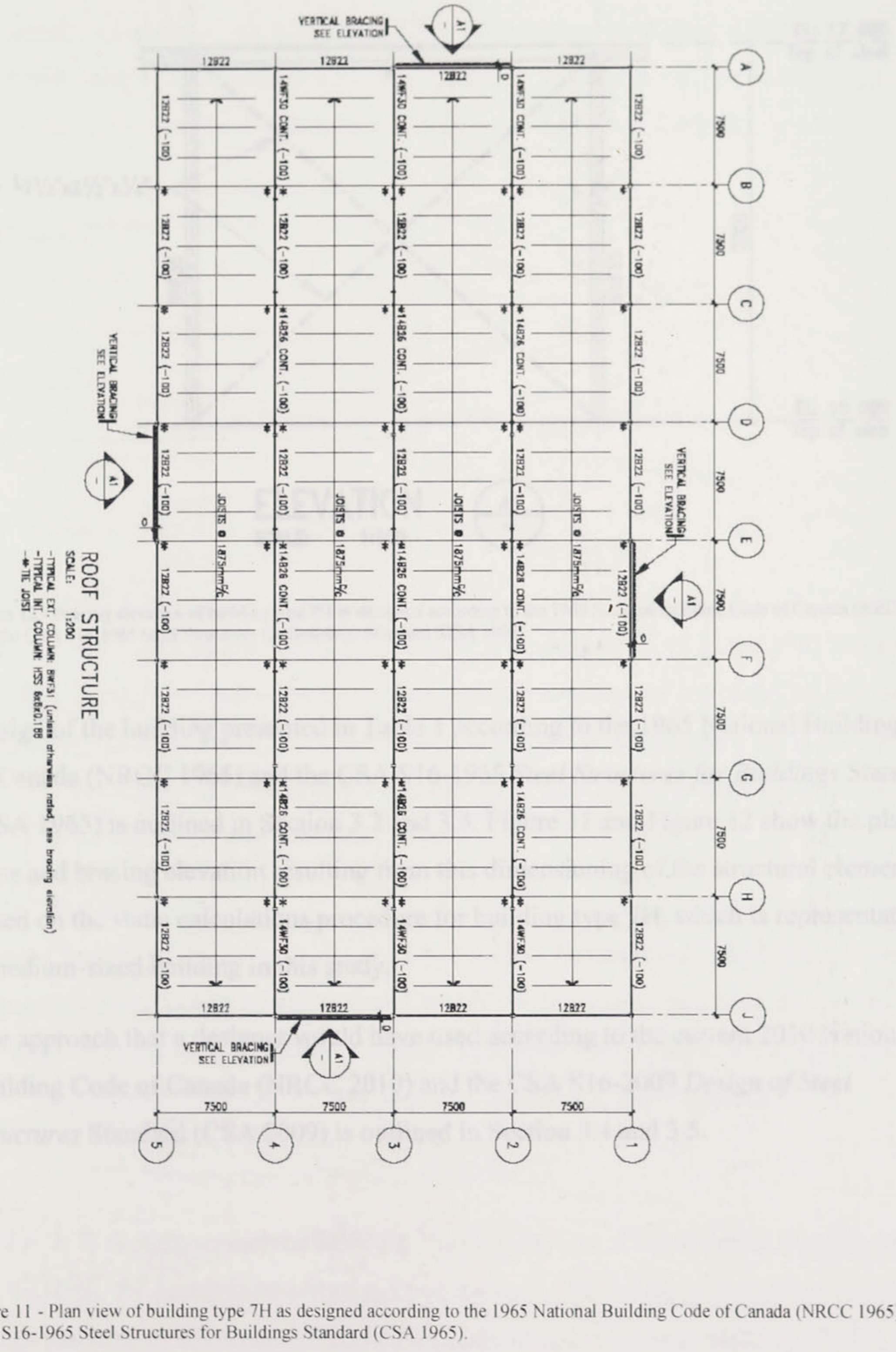


Figure 11 - Plan view of building type 7H as designed according to the 1965 National Building Code of Canada (NRCC 1965) and the CSA S16-1965 Steel Structures for Buildings Standard (CSA 1965).

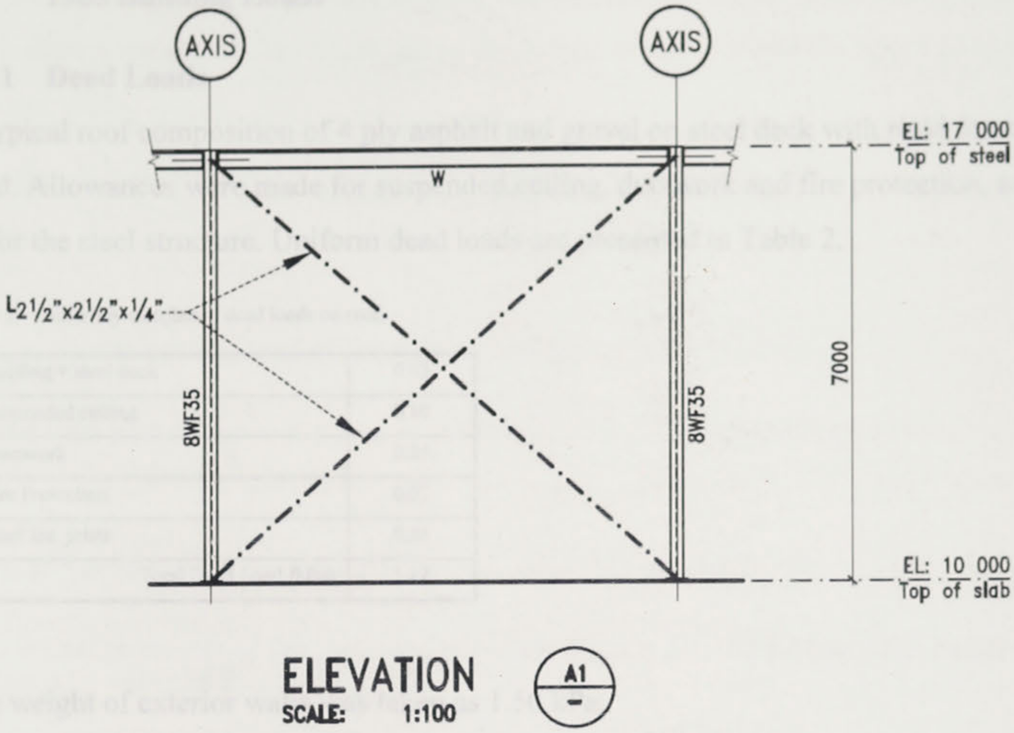


Figure 12 - Bracing elevation of building type 7H as designed according to the 1965 National Building Code of Canada (NRCC 1965) and the CSA S16-1965 Steel Structures for Buildings Standard (CSA 1965).

Design of the building presented in Table 1 according to the 1965 National Building Code of Canada (NRCC 1965) and the CSA S16-1965 *Steel Structures for Buildings* Standard (CSA 1965) is outlined in Section 3.2 and 3.3. Figure 11 and Figure 12 show the plan view and bracing elevation resulting from this dimensioning of the structural elements based on the static calculations procedure for building type 7H, which is representative of a medium-sized building in this study.

The approach that a designer would have used according to the current 2010 National Building Code of Canada (NRCC 2010) and the CSA S16-2009 *Design of Steel Structures* Standard (CSA 2009) is outlined in Section 3.4 and 3.5.

3.2 1965 Building Loads

3.2.1 Dead Loads

A typical roof composition of 4 ply asphalt and gravel on steel deck with rigid foam was used. Allowances were made for suspended ceiling, ductwork and fire protection, as well as for the steel structure. Uniform dead loads are presented in Table 2.

Table 2 - Uniformly distributed dead loads on roof.

Roofing + steel deck	0.45
Suspended ceiling	0.10
Ductwork	0.25
Fire Protection	0.07
Steel inc. joists	0.25
Total Dead Load (kPa)	1.12

The weight of exterior walls was taken as 1.50 kPa.

3.2.2 Snow Loads

The roof snow load was based on Clause 4.1.3.7 through 4.1.3.10 of the 1965 National Building Code of Canada. The prescribed ground snow load of 2.16 kPa [45 psf] was reduced by the basic snow load coefficient, C_a, of 0.80 as per Clause 4.1.3.9 since the building is not assured to be exposed to wind on all four sides. As such the uniform design snow load used was 1.73 kPa [36 psf].

3.2.3 Seismic Loads

As per Chart 10 of the Supplement no. 1 to the 1965 National Building Code of Canada, Halifax is in earthquake intensity zone 2 which corresponds to moderate damage to buildings during a seismic event. Calculation of base shear is included from Clause 4.1.3.15 through 4.1.3.17 of the 1965 National Building Code of Canada.

$$V = K \cdot W \qquad [3\text{-}1]$$

Where W is the total weight of building, due to materials of construction incorporated in the building, the design load resulting from the use of the building for storage and the design load due to the weight of service equipment and machinery (NRCC 1965).

For the one storey buildings considered in this study, there are no floors above grade so the building weight, W, is considered as the weight of the roof and the upper portion of exterior walls:

$$W= q_{DL,roof} \cdot A_{roof} + q_{DL,walls} \cdot P_{roof} \cdot (0.5\ h_{roof}) \qquad [3\text{-}2]$$

where:

$q_{DL,roof}$ = uniform dead load of the roof as outlined in Section 3.2.1;

A_{roof} = total area of the roof for each of the typical parametric buildings defined in Table 1;

$q_{DL,walls}$ = dead load of the exterior walls as defined in Section 3.2.1;

P_{roof} = total perimeter of the roof for each of the typical parametric buildings defined in Table 1;

h_{roof} = total height of the roof for each of the typical parametric buildings defined in Table 1.

Parameter K is defined in Clause 4.1.3.15(3) of the 1965 National Building Code of Canada as:

$$K=R \cdot C \cdot I \cdot F \cdot S \qquad [3\text{-}3]$$

where:

R = 2 for earthquake intensity zone;

C = 1.25 for normal construction;

I = 1.0 for normal importance;

F = 1.0 for normal foundation conditions;

S = 0.025 for a one –storey building.

For a one-storey building the horizontal shear at the roof level is equal to the base shear, V, and overturning moment is equal to:

$$M = V \cdot h_{roof} \qquad [3\text{-}4]$$

3.2.4 Wind Loads

As per Chart 8 of the Supplement no. 1 to the 1965 National Building Code of Canada, Halifax has a wind gust speed, G, of 90 mph. The velocity pressure, P, is defined as:

$$P = C \cdot G^2 = 1.05 \text{ kPa } [21.87 \text{ psf}] \qquad [3\text{-}5]$$

Where C is a constant dependent on air temperature and atmospheric pressure taken as 0.0027 as suggested in Supplement no. 1 (NRCC 1965).

Calculation of wind pressure is included from Clause 4.1.3.11 through 4.1.3.13 of the 1965 National Building Code of Canada.

$$F = P \cdot C_h \cdot C_p \cdot A \qquad [3\text{-}6]$$

For one storey buildings with heights ranging from 4m to 10m [13.12ft to 32.8ft] the coefficient with respect to variation in height, C_h, is 1.00 as per Table 4.1.3.D. The pressure factor, C_p, is 0.85 (NRCC 1965).

For a one-storey building the horizontal shear at the roof level is calculated using an area equal to one half the height multiplied by the width or length of the building, depending on the wind direction under consideration.

3.2.5 Load Combinations

As per the 1965 National Building Code of Canada the load combinations used for dimensioning of the typical design buildings are presented in Table 3.

Table 3 - Load combinations (NRCC 1965).

1. 1.0 DL + 1.0 SL
2a. 1.0 DL + 1.0 WL
2b. 1.0 DL + 1.0 EL
3a. 0.75 (DL + SL + WL)
3b. 0.75 (DL + SL + EL)

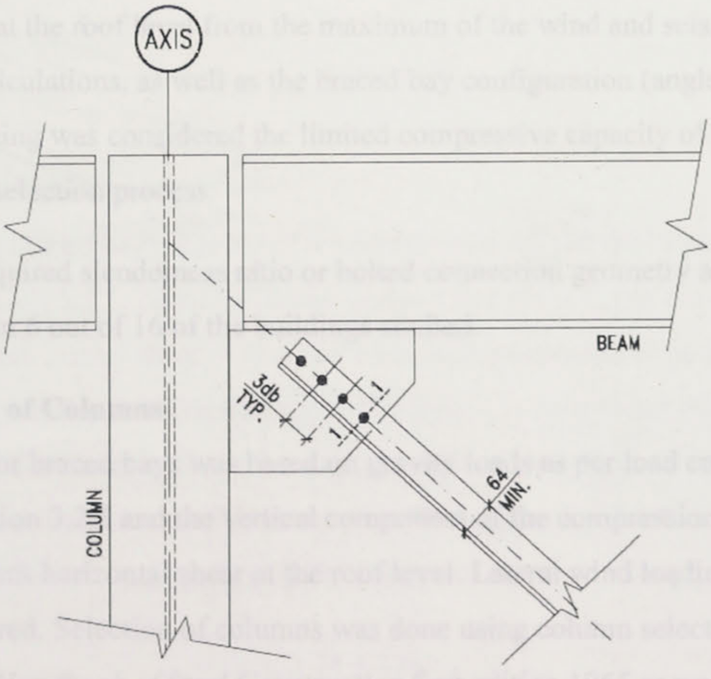
3.3 Design of Braced Bay According to 1965 CSA-S16 Standard

3.3.1 Selection of Braces

A tension only braced frame design using single angles was selected as was typical of a 1965 building design for one storey buildings. A307 bolts were used in order to determine typical connection sizes and the appropriate net area of the section. Tension resistance was calculated as specified in S16-1965 Clause 16.3.1 as $0.60F_y$ with a maximum of 0.50 times the specified minimum tensile strength:

$$T_r = \min\begin{Bmatrix} 0.60 + {}^{1}/_{3}\,(0.60) = 0.80\ A_nF_y \\ 0.50 + {}^{1}/_{3}\,(0.50) = 0.67A_nF_u \end{Bmatrix} \qquad [3\text{-}7]$$

Net area was calculated considering bolted connections and taking into account the product of thickness and net width, calculated normal to the axis of the member, as specified in S16-1965 Clause 15.2. One inch bolts were assumed with a hole size 1/16 in larger, as specified in S16-1965 Clause 15.2. The width of the connected leg of angle was dimensioned in order to accommodate the bolted connection in either a single or double row of bolts as per Figure 13.



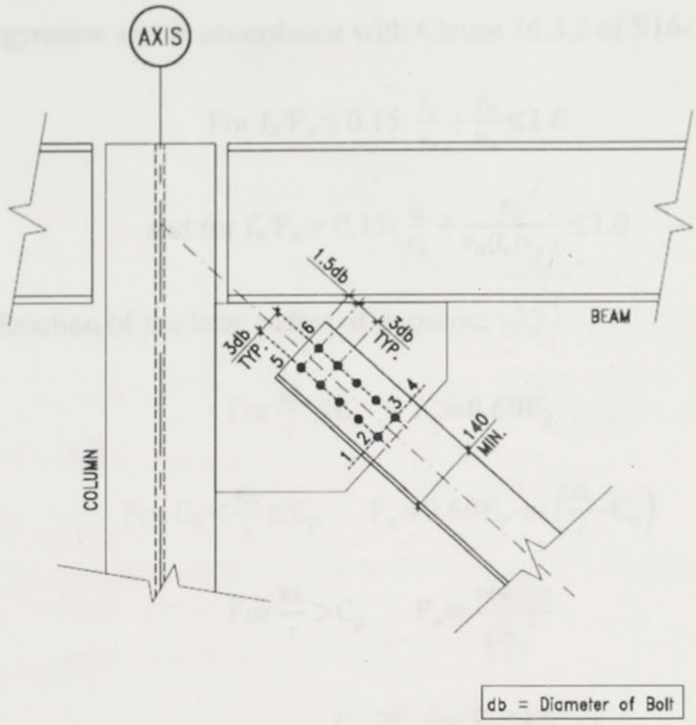


Figure 13 - Layout of typical connections, based on one row of bolts or two, assumed in dimensioning of bracing angles to account for width of leg required to permit bolting.

The tension in the brace considered for member selection was based on the value of the horizontal shear at the roof level from the maximum of the wind and seismic loads from the 1965 code calculations, as well as the braced bay configuration (angle of brace). Since tension only bracing was considered the limited compressive capacity of the brace is neglected in the selection process.

Satisfying the required slenderness ratio or bolted connection geometry affected selection of brace section in 6 out of 16 of the buildings studied.

3.3.2 Selection of Columns

Column design for braced bays was based on gravity loads as per load combinations described in Section 3.2.5 and the vertical component of the compression/tension load based on maximum horizontal shear at the roof level. Lateral wind loading out-of plane was also considered. Selection of columns was done using column selection tables as presented in the *Handbook of Steel Construction* first edition 1965 as was common design

practise. The selection tables are based on the effective length, kL, with respect to the least radius of gyration and in accordance with Clause 16.3.2 of S16-1965.

$$\text{For } f_a/F_a \leq 0.15: \frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0 \qquad [3\text{-}8]$$

$$\text{and for } f_a/F_a > 0.15: \frac{f_a}{F_a} + \frac{f_b}{F_b(f_a/F_e')} \leq 1.0 \qquad [3\text{-}9]$$

Where F_a is a function of the least radius of gyration:

$$\text{For } \frac{KL}{r} \leq C_o \qquad F_a = 0.60F_y \qquad [3\text{-}10]$$

$$\text{For } C_0 < \frac{KL}{r} \leq C_p \qquad F_a = 0.60F_y - m\left(\frac{KL}{r} - C_o\right) \qquad [3\text{-}11]$$

$$\text{For } \frac{KL}{r} > C_p \qquad F_a = \frac{149{,}000}{\left(\frac{KL}{r}\right)^2} \qquad [3\text{-}12]$$

$$\text{Where } C_o = \begin{Bmatrix} 20 \text{ for } F_y \leq 50 \\ \left(30 - \frac{F_y}{5}\right) \text{ for } F_y > 50 \end{Bmatrix} \qquad [3\text{-}13]$$

$$\text{And } C_p = \frac{535}{\sqrt{F_y - 13}} \qquad [3\text{-}14]$$

$$m = \frac{6.77 + 0.079F_y}{C_p - C_o} \qquad [3\text{-}15]$$

3.3.3 Selection of Beams

Beam design for braced bays was based on gravity loads as per the load combinations described in Section 3.2.5. Axial loads due to the effect of the beam acting as a chord or collector due to horizontal shear at the roof level was not considered as it was not common design practise in 1965. Selection of beams was done using selection tables as presented in the *Handbook of Steel Construction* first edition 1965 as was common design practise. The selection tables are based on allowable moment resistance calculated and in accordance with Clause 16.3.4 of S16-1965 based on:

$$\frac{f_b}{F_b} \leq 1.0 \qquad [3\text{-}16]$$

F_b is dependent on the minimum of the capacity in the tension flange F_{bt}, versus the compression flange, F_{bc}. For compact I-Type sections according to Clause 16.3.4.1 of S16-1965:

$$F_{bt}=0.66F_y \qquad [3\text{-}17]$$

$$F_{bc}=\max\left\{\begin{matrix}\min\left(\frac{12{,}000\ A_{fc}}{Ld}\ \text{and}\ 0.66F_y\right)\\ \min\left(\begin{matrix}0.66F_y\left(1.18\text{-}0.00091\sqrt{F_y}\cdot\frac{L}{r}\right)\\ \min\left(0.66F_y\ \text{and}\ \frac{149{,}000}{(L/r)^2}\right)\end{matrix}\right)\end{matrix}\right\} \qquad [3\text{-}18]$$

For non-compact I-Type sections according to Clause 16.3.4.4 of S16-1965:

$$F_{bt}=0.60F_y \qquad [3\text{-}19]$$

$$F_{bc}=\max\left\{\begin{matrix}\min\left(\frac{12{,}000\ A_{fc}}{Ld}\ \text{and}\ 0.60F_y\right)\\ \min\left(\begin{matrix}0.60F_y\left(1.30\text{-}0.0010\sqrt{F_y}\cdot\frac{L}{r}\right)\\ \min\left(0.60F_y\ \text{and}\ \frac{149{,}000}{(L/r)^2}\right)\end{matrix}\right)\end{matrix}\right\} \qquad [3\text{-}20]$$

The beam sections chosen were always those of least weight as presented in bold in the beam selection tables.

3.3.4 Selection of Roof Deck/Diaphragm

The roof framing system considered consisted of open-web steel joists supported on Gerber beams, as was common practise in the 1960s. Perimeter members consisted of simply supported beams.

The roof deck thickness was based on providing an adequate capacity to resist gravity loads as per load combinations in the 1965 NBCC described in Section 3.2.5. A typical deck was chosen as was common in building design in the 1960s: 22 gauge (0.76 mm thick) – 38 mm deep deck with ribs at 914mm and 6-152 mm flutes. Cold-formed steel from ASTM A 653M SS Grade 230 was considered with a nominal yield strength of F_y = 230 MPa and tensile stress of F_u = 310 MPa. For a roof deck with a triple span this deck

can be used for joist spacing's up to 2250 mm considering the dead and snow loads as previously defined in Sections 3.2.1 and 3.2.2. The fastener pattern chosen was a button punch side-lap at either 600 mm or 300 mm and 19mm welds on supports at 300 mm or 150 mm (pattern 914/4 or 914/7). The various building configurations as outlined in Table 1 had joist spacings that varied from 1.667 m to 2.025 m depending on the bay size, as shown in Table 4.

Table 4 - Joist spacing used for each design building for diaphragm design.

Building type	Bay size (m)	Joist spacing (m)
1H	6.124	2.041
2H	5	1.667
3H	5.774	1.925
4H	7.746	1.937
5H	7.071	1.768
6H	8.66	1.732
7H	7.5	1.875
8H	6.708	1.677
9H	7.825	1.956
10H	7.454	1.864
11H	7.746	1.937
12H	8.66	1.732
13H	8.101	2.025
14H	8.819	1.764
15H	9.165	1.833
16H	10.247	1.708

The fastener pattern was taken as uniform over each building's length. The strength and stiffness of the diaphragm were determined using the SDI diaphragm catalog published by the Canadian Sheet Steel Building Institute (CSSBI 2006) as presented in Table 5.

For diaphragm stiffness, G', less than 2.5 kN/mm the roof can be considered as flexible, whereas for G' between 2.5 kN/mm and 17.5 kN/mm they act more as semiflexible diaphragms (Tremblay et al. 1996).

Table 5 - Strength and stiffness of the diaphragm from SDI diaphragm catalog published by the Canadian Sheet Steel Building Institute (CSSBI).

Building type	v_f max deck (N/mm)	Sidelap spacing (mm)	Support fixation pattern	v_R (N/mm)	G' (10^3 N/mm)
1H	1.2	600	36/4	3.68	2.86
2H	1.7	600	36/4	4.37	2.61
3H	2.7	600	36/4	3.85	2.78
4H	3.9	300	36/4	4.63	2.79
5H	2.0	600	36/4	4.16	2.68
6H	2.5	600	36/4	4.24	2.65
7H	3.1	600	36/4	3.95	2.75
8H	4.5	300	36/4	5.15	2.62
9H	2.5	600	36/4	3.79	2.80
10H	3.2	600	36/4	3.97	2.74
11H	3.8	300	36/4	4.43	2.86
12H	5.0	600	36/7	5.83	9.25
13H	2.9	600	36/4	3.70	2.85
14H	3.7	600	36/4	4.17	2.68
15H	4.5	300	36/4	4.83	2.72
16H	5.6	600	36/7	5.91	9.26

3.4 2010 Building Loads

3.4.1 Dead Loads

Dead loads used in 1965 design as per Section 3.2.1 would be the same as those taken for a current design approach.

3.4.2 Snow Loads

The National Building Code of Canada 2010 prescribed snow loads are larger than those specified in the 1965 building code for Halifax. The roof snow load is currently based on Clause 4.1.6 of the National Building Code of Canada.

$$S = I_S\,[S_S(C_b \cdot C_w \cdot C_S \cdot C_a) + S_r] \qquad [3\text{-}21]$$

where:

I_s = the importance factor for snow load taken as 1.0 for normal category buildings from Table 4.1.6.2;

S_s = the prescribed 1-in-50 year ground snow load from Appendix C - Tables of climatic information [1.90 kPa];

C_b = basic roof snow load factor based on characteristic length of the roof, l_c, for large roofs where w is the width of the building and l is the length;

$$l_c=2w-(w^2/l) \quad [3\text{-}22]$$

$$\text{if } l_c \geq 70 \text{ m and } C_w=1.0\text{:} \quad C_b=1.0-(30/l_c)^2 \quad [3\text{-}23]$$

C_w = wind exposure factor taken as 1.0 for sheltered locations;

C_s =slope factor taken as 1.0 for flat roof;

C_a = shape factor taken as 1.0 for flat roof with no sources of accumulation;

S_r = the prescribed 1-in-50 year associated rain load from Appendix C - Tables of climatic information [0.60 kPa].

Table 6 - Snow loads as calculated per Clause 4.1.6 of the National Building Code of Canada 2010.

Building type	L (m)	W (m)	l_c (m)	C_b	S (kPa)
1H	24.5	24.5	25	0.80	2.12
2H	30	20	27	0.80	2.12
3H	34.6	17.3	26	0.80	2.12
4H	38.7	15.5	25	0.80	2.12
5H	42.4	42.4	42	0.80	2.12
6H	52	34.6	46	0.80	2.12
7H	60	30	45	0.80	2.12
8H	67.1	26.8	43	0.80	2.12
9H	54.8	54.8	55	0.80	2.12
10H	67.1	44.7	60	0.80	2.12
11H	77.5	38.7	58	0.80	2.12
12H	86.6	34.6	55	0.80	2.12
13H	64.8	64.8	65	0.80	2.12
14H	79.4	52.9	71	0.82	2.16
15H	91.7	45.8	69	0.80	2.12
16H	102.5	41	66	0.80	2.12

The 1965 snow load was constant for all buildings types at 1.73 kPa. Table 6 outlines the snow loads for each building configuration as per the National Building Code of Canada 2010. In general, the buildings have characteristic lengths lower than 70m, so the uniform snow load is 2.12 kPa for most of the layouts. This represents, however, an increase of 22.5% in snow loading prescribed by the National Building Code of Canada from 1965 to 2010.

3.4.3 Seismic Loads

The National Building Code of Canada 2010 prescribed earthquake loads using the equivalent static force procedure are currently based on Clause 4.1.8. The Design Base Shear is calculated as follows:

$$V = S(T_a) W \frac{M_v I_E}{R_d R_o} \qquad [3\text{-}24]$$

$$\text{With } V_{min} = S(2.0) W \frac{M_v I_E}{R_d R_o} \qquad [3\text{-}25]$$

$$\text{And for } R_d \geq 1.5 \;\; V_{max} = {}^{2}/_{3}\, S(0.2) W \frac{I_E}{R_d R_o} \qquad [3\text{-}26]$$

where:

$S(T_a)$ = the spectral acceleration at the fundamental building period;

T_a = the fundamental lateral period of the building [s]. For braced frame: $T_a = 0.025h_n$ or $T_a = 0.050h_n$ if verified by dynamic analysis;

W = total weight of the structure [kN], and includes the dead load (including partitions at 0.5 kPa), 25% of the design snow load, 60% of storage live loads, and full contents of any tanks;

M_v = higher mode participation factor – For Halifax ($S_a(0.2)/S_a(2.0)$ =12.1) and braced frame M_v is 1.0 for $T_a \leq 1.0$ and 1.5 for $T_a \geq 2.0$ with linear interpolation for values in between;

I_E = the importance factor for earthquake load taken as 1.0 for normal category buildings from Table 4.1.6.2;

R_d = ductility-related force modification factor, taken as 1.5 for conventional steel construction;

R_o = overstrength-related force modification factor, taken as 1.3 for conventional steel construction.

In general, although dead loads are constant, the weight to consider in seismic design, W, is greater for all building types according to the National Building Code of Canada 2010 due to the inclusion of 25% of the design snow load in the current design practise. No snow load is specified in W, as per the 1965 NBCC.

3.4.4 Wind Loads

The National Building Code of Canada 2010 prescribed wind loads are currently based on Clause 4.1.7. External wind pressure, p, is calculated as follows:

$$p=I_w \cdot q \cdot C_e \cdot C_g \cdot C_p \qquad [3\text{-}27]$$

where:

I_w = the importance factor for wind load taken as 1.0 for normal category buildings from Table 4.1.6.2;

q = the prescribed 1-in-50 year wind load from NBCC 2010 Appendix C - tables of climatic information [0.40 kPa];

C_e = exposure factor, taken as:

$$C_e=(h_n/10)^{0.2} \qquad [3\text{-}28]$$

C_g and C_p = gust effect factor and external pressure coefficient respectively, taken from Figure I-7 of National Building Code Commentary for low rise buildings with $h_n \leq 20$ m.

Table 7 outlines the wind loads for each building configuration as per the National Building Code of Canada 2010.

Table 7 - Wind loads as calculated per Clause 4.1.7 of the National Building Code of Canada 2010.

Building type	L (m)	W (m)	H (m)	C_e	End-zone z (m)	$p_{s\ end\ zone}$ (kPa)	$p_{s\ mid\ zone}$ (kPa)	V in L (kN)	V in W (kN)
1H	24.5	24.5	4	0.83	1.60	0.65	0.43	45	45
2H	30	20	5	0.87	2.00	0.68	0.45	72	50
3H	34.6	17.3	6	0.90	1.73	0.70	0.47	102	54
4H	38.7	15.5	7	0.93	1.55	0.73	0.48	136	58
5H	42.4	42.4	5	0.87	2.00	0.68	0.45	100	100
6H	52	34.6	6	0.90	2.40	0.70	0.47	153	104
7H	60	30	7	0.93	2.80	0.73	0.48	213	111
8H	67.1	26.8	8	0.96	2.68	0.75	0.50	278	117
9H	54.8	54.8	6	0.90	2.40	0.70	0.47	161	161
10H	67.1	44.7	7	0.93	2.80	0.73	0.48	237	161
11H	77.5	38.7	8	0.96	3.20	0.75	0.50	321	167
12H	86.6	34.6	9	0.98	3.46	0.76	0.51	413	174
13H	64.8	64.8	7	0.93	2.80	0.73	0.48	229	229
14H	79.4	52.9	8	0.96	3.20	0.75	0.50	329	223
15H	91.7	45.8	9	0.98	3.60	0.76	0.51	437	226
16H	102.5	41	10	1.00	4.00	0.78	0.52	554	234

3.4.5 Load Combinations

As per the 2010 National Building Code of Canada the load combinations are presented in Table 8. Unlike the 1965 code, the 2010 combinations are based on factored loading to be used in conjunction with limit states design.

Table 8 - Load combinations (NRCC 2010).

NBCC 2010 Load Combinations		
	Principle Loads	Companion Loads
1	1.4D	
2	(1.25D or 0.9D) + 1.5L	0.5S or 0.4W
3	(1.25D or 0.9D) + 1.5S	0.5L or 0.4W
4	(1.25D or 0.9D) + 1.4W	0.5L or 0.5S
5	1.0D + 1.0E	0.5L + 0.25S

3.5 Design of Braced Bay According to 2009 CSA-S16 Standard

3.5.1 Selection of Braces

Using the same principle of tension only braced frame with a bolted connection design, tension resistance using CSA-S16-09 Clause 13.2 is calculated as:

$$T_r = \min\left\{\begin{array}{c} \phi A_g F_y \\ \phi_u\left[U_t A_n F_u + 0.60 A_{gv}\frac{(F_y+F_u)}{2}\right] \\ \phi_u A_{ne} F_u \end{array}\right\} \quad [3\text{-}29]$$

$$\text{and } 0.75\phi A_n F_y \text{ for pinned connections}$$

CSA-S16-09 includes tension resistance calculations accounting for shear lag and block shear tear out, as well as a calculation using the gross section which was not present in 1965 code.

CSA-S16-09 Clause 12 defines gross and net areas. The major differences between the 1965 and 2009 Standards are the inclusion of A_{ne} to account for shear lag, and A_{gv} for the gross area in shear. Whereas, in 1965 Standard net width was calculated normal to the axis of the member with both single or double row of bolts, calculation of net area as per the 2009 Standard would include additional fracture paths to account for tension and shear block failure. This may result in sections chosen according to 1965 code to fail 2009 design criteria with regards to the connection design.

3.5.2 Selection of Columns

Compression resistance using CSA-S16-09 Clause 13.3 is calculated, for doubly symmetric shapes, as:

$$C_r = \phi A F_y (1+\lambda^{2n})^{-1/n} \quad [3\text{-}30]$$

$$\text{with } \lambda = \frac{kL}{r}\sqrt{\frac{F_y}{\pi^2 E}} \text{ and } n=1.34 \text{ for hot-rolled shapes} \quad [3\text{-}31]$$

For out of plane bending on laterally unsupported members due to wind loads Clause 13.6 of CSA-S16-09, as well as the combined bending/compression formulas in Clause 13.8 would be used according to current design procedures.

3.5.3 Selection of Beams

Compression resistance for beams acting as chords and collectors using CSA-S16-09 Clause 13.23 is calculated, for doubly symmetric shapes, as per Equation 3-30.

Bending resistance for laterally supported members using CSA-S16-09 Clause 13.5 is calculated as:

$$M_r = \phi Z F_y \text{ for Class 1 and 2 sections} \quad [3\text{-}32]$$

$$M_r = \phi S F_y \text{ for Class 3 sections} \quad [3\text{-}33]$$

According to current design practise beam design for braced bays would be based on gravity loads as per load combinations described in Section 3.4.5, as well as axial loads due to effect of chord and collector loading from horizontal shear at the roof level. The combined bending/compression formulas in using CSA-S16-09 Clause 13.8 would be used by designers for member selection.

3.5.4 Selection of Roof Deck/Diaphragm

Selection of the roof deck based on gravity loads would have been the same for both the 1965 and 2010 design codes (22 gauge (0.76 mm thick) – 38 mm deep deck with ribs at 914 mm, trapezoidal profile with flutes at 152 mm on center). Diaphragm design, however, may differ as CSA S16-09 requires that diaphragms and connections of primary framing members be designed using a multiplier of R_d =1.50 in Section 27.11 for conventional construction when $I_E F_a S_a(0.2)$ exceeds 0.45. Therefore the factored shear to consider would be 1.5 times the 1965 value which could potentially result in the selection of different fastener patterns from the SDI diaphragm catalog published by the Canadian Sheet Steel Building Institute (CSSBI 2006) than those presented in Table 5.

3.6 Selection of Earthquake Records

A series of ten historical and ten artificial records were selected for use with the one-storey buildings in this study based on guidelines for selection and scaling of time histories as presented by Atkinson (2009). Earthquake records were scaled such that their

response spectrums were matched to the uniform hazard spectrum presented in the NBCC 2010 for Halifax for Class C soil conditions, and for the 2% in 50 year return period prescribed in the 2010 NBCC.

Artificial records were selected based on recommendations by Atkinson (2009) to use records near the large end of the distance range for each magnitude for eastern Canadian sites of low seismicity. This implies the use of scaled down magnitude 6 at fault-distance range 20 to 30 km (M6 set 2) and magnitude 7 at fault-distance range 50 to 100 km (M7 set 2). Artificial records used were selected from a national database of earthquake records published by Atkinson (2009).

The ten records selected were based on consideration of minimizing standard deviation and recommendations on appropriate scaling factors as outlined in Atkinson (2009) as presented in Table 9.

Table 9 - Ten artificial records and scaling factors based on Atkinson (2009).

M	Record No.	Scaling factor
6C2	1	0.438
6C2	3	0.697
6C2	5	0.600
6C2	9	0.468
6C2	15	0.668
7C2	1	0.586
7C2	2	0.706
7C2	3	0.643
7C2	7	0.654
7C2	8	0.790

The percentile mean of the spectral accelerations for these artificial scaled records as compared to the uniform hazard spectrum for Halifax is presented in Figure 14.

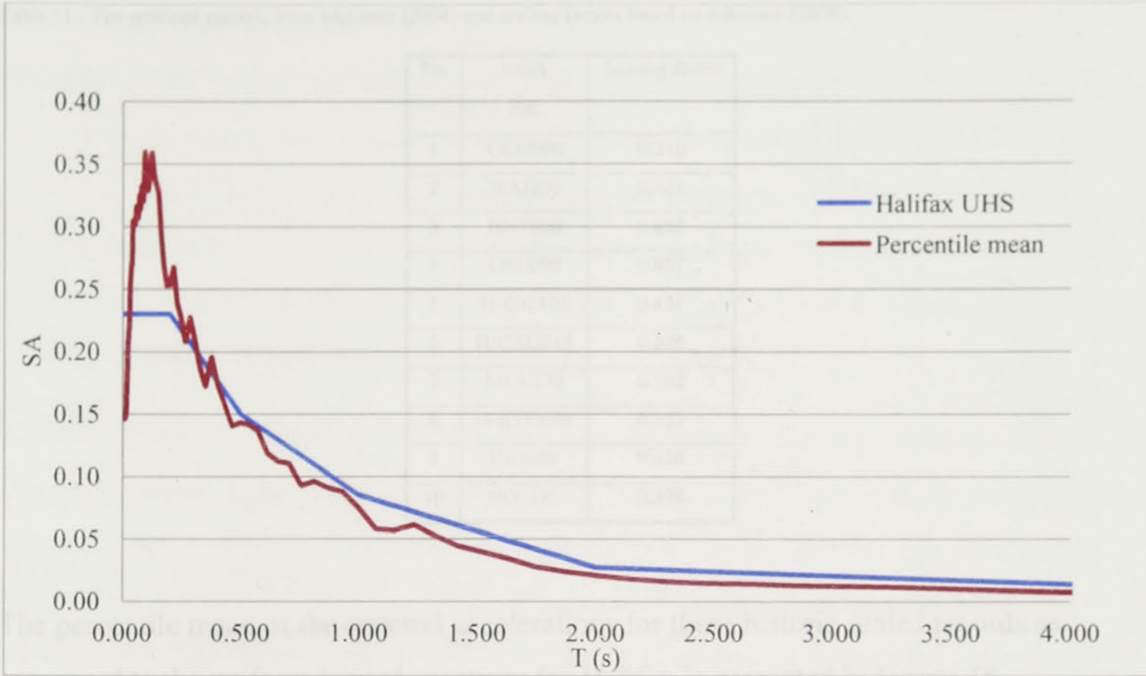


Figure 14 - Spectral acceleration vs. time for percentile mean of the scaled artificial records as compared with UHS for Halifax.

Historical records were selected from a database of earthquake records by McGuire (2004). The same ten records as used by Caruso-Juliano (2012) for eastern Canada (Montreal) were chosen and are presented in Table 10. The historic earthquake records were scaled such that their response spectra were matched to the uniform hazard spectrum presented in the NBCC for Halifax, as recommended by Atkinson (2009). The scaling factors are presented in Table 11.

Table 10 - Ten historical records based from McGuire (2004).

No	NGA No.	Event	M_W	Station
1	CCN090	Jan. 17, 1994 Northridge	6.7	LA-Century City CC North
2	WAI290	Jan. 17, 1994 Northridge	6.7	Huntington Beach Waikiki
3	HNT000	Jan. 17, 1994 Northridge	6.7	Huntington Beach Lake St
4	DEL090	Jan. 17, 1994 Northridge	6.7	Lakewood Del Amo Bvld
5	H-E01140	Oct 15, 1979 Imperial Valley	6.5	El Centro Array #1
6	H-CXO315	Oct 15, 1979 Imperial Valley	6.5	Calexico Fire Station
7	MUL279	Oct. 1, 1987 Whittier Narows	6.0	Beverly Hills - 14145 Mulhol
8	A-STC090	Oct. 1, 1987 Whittier Narows	6.0	Northridge- 17645 Saticoy St
9	IND000	June 28, 1992 Landers	7.3	Indio-Coachella Canal
10	HOS180	June 28, 1992 Landers	7.3	San Bernardino-E&Hospitality

Table 11 - Ten artificial records from McGuire (2004) and scaling factors based on Atkinson (2009).

No	NGA No.	Scaling factor
1	CCN090	0.210
2	WAI290	0.422
3	HNT000	0.652
4	DEL090	0.467
5	H-E01140	0.451
6	H-CXO315	0.306
7	MUL279	0.502
8	A-STC090	0.357
9	IND000	0.436
10	HOS180	0.398

The percentile mean of the spectral accelerations for these historic scaled records as compared to the uniform hazard spectrum for Halifax is presented in Figure 15.

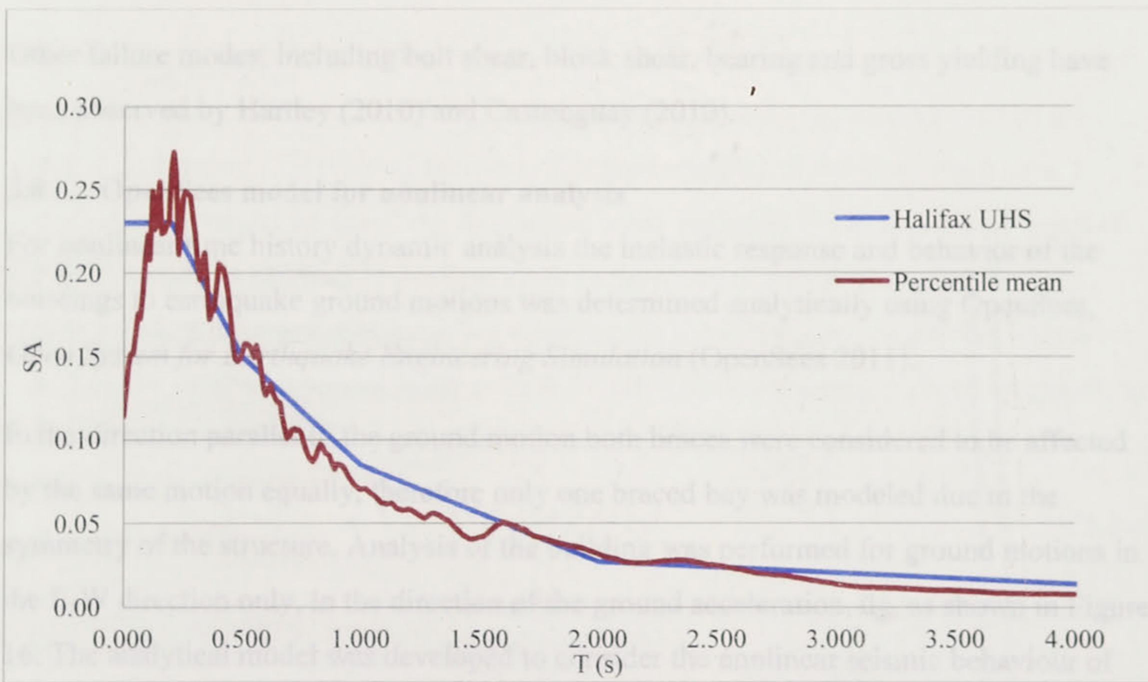


Figure 15 - Spectral acceleration vs. time for percentile mean of the scaled historic records as compared with UHS for Halifax.

3.7 CBF Test Program

Testing of CBF braces and their connections was conducted by Caruso-Juliano (2012) on double-angle braces which were extracted from the PMQ Sector 4 Rio Tinto facility (1967) in Sorel-Tracy, Québec. These braces were separated such that 7 single and 6 double angle tests were performed, with the remaining angles reserved for coupon tests. The testing protocol was taken from FEMA 461: Interim Protocol I – Quasi-Static Cyclic Testing (FEMA 2007) (Caruso-Juliano 2012).

Caruso-Juliano (2012) observed that net section fractures of varying ductility occurred in the braces tested as the dominant failure mode. As well, there was no gross yielding over the entire brace length, only concentrated plasticity in the net-section. The most brittle net-section fracture occurred at 0.39% elongation of the length of the brace, but elongations were observed up to 1.85% for the existing braces. Retrofitted braces were also tested where axial elongations were seen up to 6.17% (Caruso-Juliano 2012).

Other failure modes, including bolt shear, block shear, bearing and gross yielding have been observed by Hartley (2010) and Castonguay (2010).

3.8 OpenSees model for nonlinear analysis

For nonlinear time history dynamic analysis the inelastic response and behavior of the buildings to earthquake ground motions was determined analytically using OpenSees, *Open System for Earthquake Engineering Simulation* (OpenSees 2011).

In the direction parallel to the ground motion both braces were considered to be affected by the same motion equally, therefore only one braced bay was modeled due to the symmetry of the structure. Analysis of the building was performed for ground motions in the E-W direction only, in the direction of the ground acceleration, üg, as shown in Figure 16. The analytical model was developed to consider the nonlinear seismic behaviour of the CBFs, including the flexibility, strength, and distributed mass of the roof diaphragm.

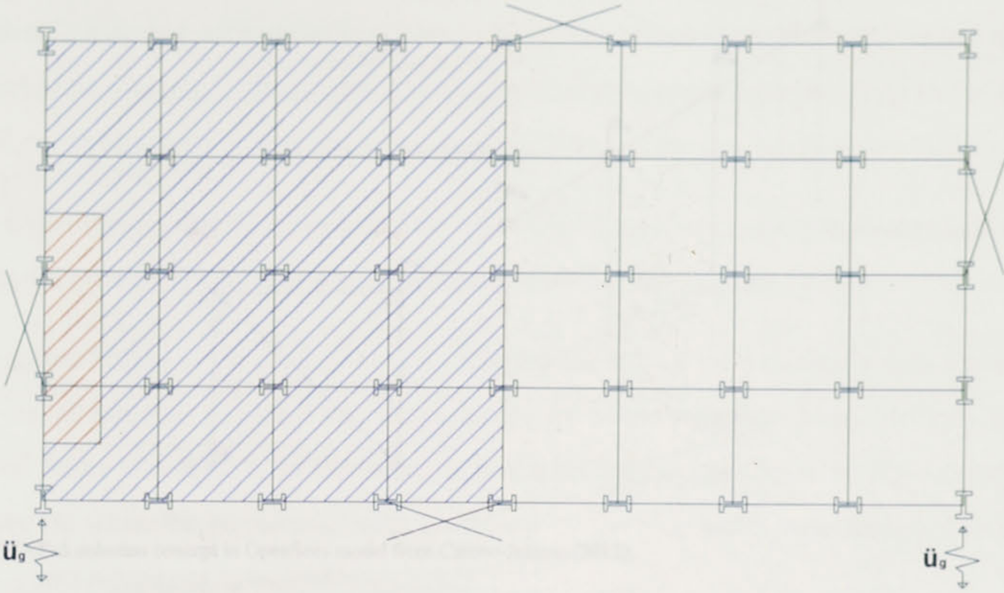


Figure 16 - Conceptual plan view of OpenSees model from Caruso-Juliano (2012).

The roof diaphragm was modelled as a flexible diaphragm using translational horizontal springs, for shear stiffness, and elastic beam-column elements connecting each of the translational springs for flexural stiffness. The shear rigidity of the diaphragm was included in the model by assigning diaphragm shear stiffness, G', as calculated based on the SDI method as outlined in Section 3.3.4, to the translational springs. The flexural rigidity of the diaphragm was included by assigning the diaphragm moment of inertia based on the exterior collector beam, which function as the diaphragm chord members, to the elastic beam-column elements (Caruso-Juliano 2012).

$$I=2A_f(d/2)^2 \qquad [3\text{-}34]$$

where A_f is the area of the collector beam flange area and d is the width of the diaphragm in the E-W direction (Medhekar et al. 1999).

Additional dummy columns with their corresponding gravity loading and attached to the diaphragm and CBF with rigid links were included to account for P-Δ effects (Medhekar et al. 1999), as shown in Figure 17. These P-Δ columns were modelled using an elastic beam-column element with infinite rigidity.

Figure 16: Conceptual plan view of OpenSees model from Caruso-Juliano (2012)

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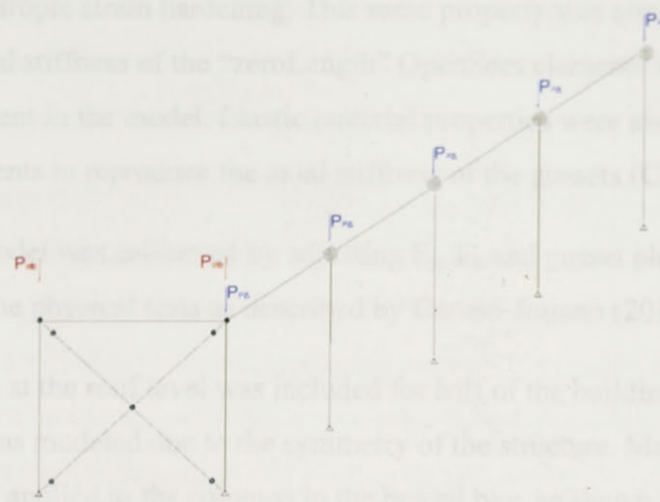

Figure 17 - P-Δ columns concept in OpenSees model from Caruso-Juliano (2012).

The lateral stiffness was provided by the diagonals of the CBF, modelled using a nonlinear beam-column element divided into fiber elements such that bi-axial bending as well as axial load and buckling effects were included. Braces were divided into 10 fibre elements across the depth and 4 across the thickness of the element, as shown in Figure 18 (Caruso-Juliano 2012).

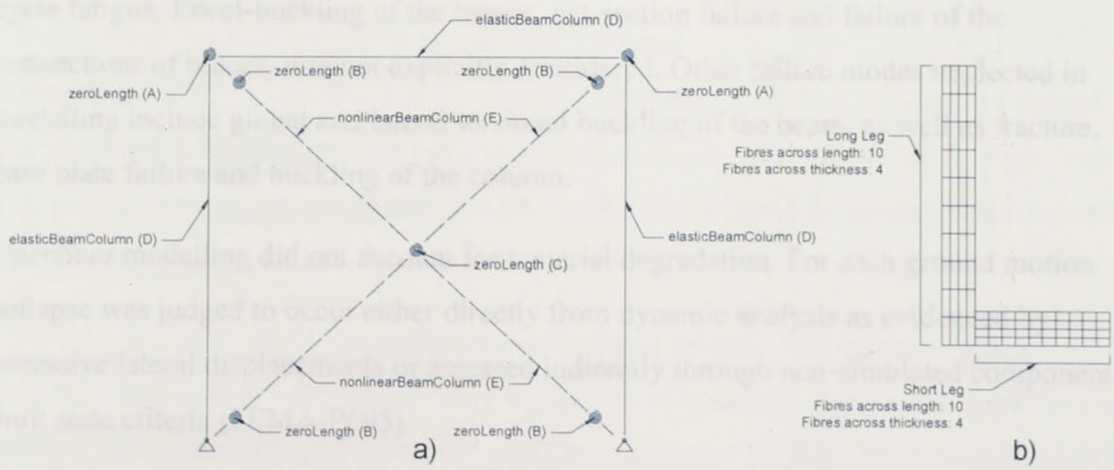

Figure 18 - a) Bracing bent spring model and b) Discretization of brace member. Caruso-Juliano (2012).

An initial out of straightness of L/500 was assigned to the middle of the brace to induce the buckling behaviour. A material property of *Steel02* was assigned to the braces which

accounted for isotropic strain hardening. This same property was also assigned to the out-of-plane rotational stiffness of the "zeroLength" OpenSees elements which made up the gusset plate element in the model. Elastic material properties were assigned to these zeroLength elements to reproduce the axial stiffness of the gussets (Caruso-Juliano 2012).

The OpenSees model was calibrated by adjusting F_y, F_u and gusset plate rotational springs using data from the physical tests as described by Caruso-Juliano (2012).

The seismic mass at the roof level was included for half of the building's area since only one braced bay was modeled due to the symmetry of the structure. Mass tributary to the braced frame was applied to the columns in the braced bay, as shown by the red hatch, in Figure 16, while the remaining mass, as shown by the blue hatch, was assigned to the P-Δ columns equally (Caruso-Juliano 2012).

Mass and stiffness proportional damping was included, with 2% critical damping considered. For the time history analysis, a Krylov Newtown algorithm was used with an integration time step of $d_t/30$.

3.8.1 Failure Criteria

Failure modes explicitly modeled using OpenSees included global buckling and low-cycle fatigue. Local-buckling of the braces, net-section failure and failure of the connections of braces, was not explicitly considered. Other failure modes neglected in modelling include global and lateral torsional buckling of the beam, as well as fracture, base plate failure and buckling of the column.

OpenSees modelling did not account for material degradation. For each ground motion collapse was judged to occur either directly from dynamic analysis as evidenced by excessive lateral displacements or assessed indirectly through non-simulated component limit state criteria (FEMA-P695).

To account for failure modes not explicitly modeled a failure criteria associated with brace elongation was used to account for connection failure in the performance evaluation process. Modes of failure considered were net section failure, bearing failure, bolt shear, and block shear failure.

accounted for isotropic strain hardening. This same property was also assigned to the out-of-plane rotational stiffness of the "zeroLength" OpenSees elements which made up the gusset plate element in the model. Elastic material properties were assigned to these zeroLength elements to reproduce the axial stiffness of the gussets (Caruso-Juliano 2012).

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The seismic mass at the roof level was included for half of the building's area since only one braced bay was modeled due to the symmetry of the structure. Mass tributary to the braced frame was applied to the columns in the braced bay, as shown by the red hatch, in Figure 16, while the remaining mass, as shown by the blue hatch, was assigned to the P-Δ columns equally (Caruso-Juliano 2012).

Mass and stiffness proportional damping was included, with 2% critical damping considered. For the time history analysis, a Krylov Newtown algorithm was used with an integration time step of 0.20.

3.8.1 Failure Criteria

Failure modes explicitly modeled using OpenSees included global buckling and low-cycle fatigue. Local-buckling of the braces, net-section failure and failure of the connections of braces, was not explicitly considered. Other failure modes neglected in modelling include global and lateral torsional buckling of the beam, as well as fracture, base plate failure and buckling of the column.

OpenSees modelling did not account for material degradation. For each ground motion collapse was judged to occur either directly from dynamic analysis as evidenced by excessive lateral displacements or assessed indirectly through non-simulated component limit state criteria (FEMA-P695).

To account for failure modes not explicitly modeled a failure criteria associated with brace elongation was used to account for connection failure in the performance evaluation process. Modes of failure considered were net section failure, bearing failure, bolt shear, and block shear failure.

Three limit state criteria for net-section fracture were used based on tests by Caruso-Juliano (2012). Four limit state criteria for bearing, block shear, bolt shear and gross yield were used based on a report by Castonguay (2010).

The ratio of ultimate deformation over length (δ_{ult}/L), converted into a percentage drift, was used to establish the limits for the seven limit state criteria previously defined. As each building had a different geometry and brace length the ratio of ultimate deformation over length (δ_{ult}/L) varied for each configuration and each limit state criteria. This ratio was also dependent on the test program, as the brace length considered in the testing was not the same as the buildings used in this study. The ratios were therefore also adjusted to take into account the elastic vs. inelastic yield length of the braces.

3.8.2 Incremental Dynamic Analysis (IDA)

An incremental dynamic analysis (IDA) method was used in order to evaluate the seismic response of the CBFs, and to obtain maximum building drifts and forces. This method uses a series of ground motions with increasing incremental intensity until failure is reached (Uriz & Mahin 2004). For Halifax the IDA was performed for 16 structures, 20 ground motions, and scaling factors from 0.2 to 6.0. Each IDA had 7 failure criteria, as described in Section 3.8.1.

3.9 Performance level and acceptance criteria

In the NBCC 2010, a building's performance under seismic loading is primarily judged by the system satisfying strength criteria as defined in CSA S16-09 and drift limits. The goal is to prevent major failure and loss of life. For buildings in the normal importance category a drift limit of 2.5% is specified in the NBCC.

Chapter 7 of FEMA P695: Quantification of building seismic performance factors outlines a methodology for performance evaluation of buildings for seismic events, which is based on establishing global seismic performance factors (FEMA 2009, NEHRP 2010).

To establish minimum acceptable performance in this study based on the usage of non-simulated failure criteria FEMA P695 methodology is employed. As well, spectral acceleration – fragility curves are used to represent probability of failure, as presented in Section 4.

For the evaluation based on FEMA methodology, the buildings as outlined in Table 1 of this report were divided into performance groups, based on common building features or behavioural characteristics. In this study the seven failure criteria as defined in Section 3.8.1 were used to define the performance groups. The study contains sixteen archetype buildings according to FEMA nomenclature (FEMA 2009, NEHRP 2010).

Acceptance criteria were based on measuring the probability of collapse using the collapse margin ratio (CMR), and comparing it to acceptable values. The collapse margin ratio (CMR) was calculated for each archetype as the ratio of the median collapse intensity, $\hat{S}_{CT}$, to the maximum considered earthquake (MCE) spectral demand, S_{MT} (FEMA 2009, NEHRP 2010):

$$\mathrm{CMR}=\frac{\hat{S}_{CT}}{S_{MT}} \qquad [3\text{-}35]$$

Since the earthquake records used in the study are based on matching the uniform hazard spectrum the maximum considered earthquake spectral demand, S_{MT}, was taken as 1.0. As such the collapse margin ratio (CMR) was equal to the median collapse intensity, $\hat{S}_{CT}$.

The collapse margin ratio (CMR) was then modified to account for the effects of spectral shape, using a spectral shape factor for each archetype. This is called the adjusted collapse margin ratio (ACMR) where for each archetype, i (FEMA 2009, NEHRP 2010):

$$\mathrm{ACMR}_i=\mathrm{SSF}_i \times \mathrm{CMR}_i \qquad [3\text{-}36]$$

The spectral shape factor, SSF, varies according to the fundamental period, T, and period-based ductility; μ_T. Values for SSF as per FEMA P-695 are presented in Table 12 (FEMA 2009, NEHRP 2010).

Table 12 - Spectral shape factor, SSF, according to the fundamental period, T, and period-based ductility, μ_T (FEMA 2009).

SSF	Period based ductility, μ_T							
T (sec.)	1.0	1.1	1.5	2	3	4	6	≥ 8
≤ 0.5	1.00	1.02	1.04	1.06	1.08	1.09	1.12	1.14
0.6	1.00	1.02	1.05	1.07	1.09	1.11	1.13	1.16
0.7	1.00	1.03	1.06	1.08	1.10	1.12	1.15	1.18
0.8	1.00	1.03	1.06	1.08	1.11	1.14	1.17	1.20
0.9	1.00	1.03	1.07	1.09	1.13	1.15	1.19	1.22
1.0	1.00	1.04	1.08	1.10	1.14	1.17	1.21	1.25
1.1	1.00	1.04	1.08	1.11	1.15	1.18	1.23	1.27
1.2	1.00	1.04	1.09	1.12	1.17	1.20	1.25	1.30
1.3	1.00	1.05	1.10	1.13	1.18	1.22	1.27	1.32
1.4	1.00	1.05	1.10	1.14	1.19	1.23	1.30	1.35
1.5	1.00	1.05	1.11	1.15	1.21	1.25	1.32	1.37

The period-based ductility, μ_T, was taken as the ratio of ultimate roof drift, Δ_u, to effective yield drift, $\Delta_{y,eff}$, from non-linear analysis (FEMA 2009, NEHRP 2010):

$$\mu_T = \Delta_u / \Delta_{y,eff} \quad , \qquad [3\text{-}37]$$

FEMA P-695 accounts for uncertainty by introducing quality ratings which are assigned for design requirements, test data, non-linear modelling, and record-to-record uncertainty. Total system collapse uncertainty, β_{TOT}, is then (FEMA 2009, NEHRP 2010):

$$\beta_{TOT} = \sqrt{B^2{}_{RTR} + B^2{}_{DR} + B^2{}_{TD} + B^2{}_{MDL}} \qquad [3\text{-}38]$$

Record-to-record uncertainty is due to variability in response to different records and accounted for using the factor β_{RTR}. For buildings with significant period elongation ($\mu_T \geq 3$) a value of β_{RTR} equal to 0.40 is recommended. For buildings with little or no period elongation ($\mu_T < 3$) the value of β_{RTR} is variable according to period-based ductility, μ_T. A factor β_{RTR} of 0.40 was assigned to this study.

Design requirement uncertainty is due to level of robustness in design requirements and accounted for using the factor β_{DR}, based on the quality of design requirements as outlined in Section 3.4 of FEMA P-695 (FEMA 2009). A factor β_{DR} of 0.30 was assigned to the design for this study.

Test data uncertainty is due to level of completeness and robustness of test data used and accounted for using the factor β_{TD}, based on the quality of test data as outlined in Section 3.6 of FEMA P-695(FEMA 2009). A factor β_{TD} of 0.45 was assigned to the test protocol used in this study based on recommendations by Caruso-Juliano (2012).

Modelling uncertainty is due to level of accuracy in modelling structural response and accounted for using the factor β_{MDL}, based on the quality of structural modelling as outlined in Section 5.7 of FEMA P-695(FEMA 2009). Test data was used to calibrate analytical models as described by Caruso-Juliano (2012) using brace subassembly tests. However, as seen in Section 3.8.1 of this report not all brace failure modes were explicitly accounted for in modelling. Also, the use of fibre element for the brace imposed a limited ability to simulate local buckling (NEHRP 2010). A factor β_{MDL} of 0.45 was assigned to the analytical models used in this study as per recommendations by Caruso-Juliano (2012).

For each performance group, the acceptable adjusted collapse margin ratio, $ACMR_{10\%}$, was determined from Table 7-3 of FEMA P-695 based on total system collapse uncertainty, β_{TOT}. Within a performance group the acceptable adjusted collapse margin ratio for a specific archetype is denoted $ACMR_{20\%}$, and determined from Table 7-3 of FEMA P-695 as well (FEMA 2009, NEHRP 2010).

For each performance group, the acceptable adjusted collapse margin ratio, $ACMR_{10\%}$, is compared to the average adjusted collapse margin ratio, ACMR, of all archetypes. This is based on limiting the probability of collapse under the maximum considered earthquake (MCE) to 10%. Within each performance group, the acceptable adjusted collapse margin ratio is relaxed to a probability of collapse of 20%, $ACMR_{20\%}$, and compared to the adjusted collapse margin ratio, ACMR, of each archetype individually. This recognises that although the average probability of collapse for a performance group must satisfy more stringent requirements, individual archetypes may exceed a 10% probability of collapse (FEMA 2009, NEHRP 2010).

Sources of error in the determination of performance criteria include the judgement used in interpreting the results of the nonlinear time history dynamic analysis, in assessing uncertainty, and in the rounding of values in the design (FEMA 2009).

3.10 Summary

Braced frame and building dimensioning was carried out using the 1965 National Building Code of Canada and S16-1965 for the buildings outlined in Table 1. A set of twenty earthquake records were selected to form the basis of non linear analysis from both artificial and historic earthquake databases. These were used with an incremental dynamic analysis and OpenSees model which was calibrated according to test results on CBFs by Caruso-Juliano (2012). Failure criteria associated with brace elongation were used in the performance evaluation process. Modes of failure considered were net section failure, bearing failure, bolt shear, and block shear failure based on testing by Caruso-Juliano (2012) and Castonguay (2010). Seven limit states criteria were established and used as the basis to establish minimum acceptable performance in this study based on FEMA P695 methodology. Acceptance criteria were based on measuring the probability of collapse using the adjusted collapse margin ratio (ACMR), and comparing it to acceptable values, $ACMR_{10\%}$ and $ACMR_{20\%}$.

Chapter 4 – Results

4.1 IDA Analysis

The results from the incremental dynamic analysis (IDA) described in Section 3.8.2 of this report are presented in Figure 19 to Figure 22 for buildings type 2H, 7H, 8H, and 16H. Maximum percent storey drift is plotted against the ground motion scaling factor for each earthquake ground motion. Building type 2H is representative of the buildings in this study on the smaller scale, while 7H and 8H are representative of medium sized buildings, and 16H is representative of buildings of a larger scale. Each of the seven failure criteria outlined in Section 3.8.1 of this report is represented by a vertical red line at the maximum storey drift corresponding to the onset of failure based on testing by Caruso-Juliano (2012) and Castonguay (2010) as numbered in Table 13. Figures for the results of the IDA for all sixteen buildings in this study are presented in Appendix D.

A scaling factor of 1 to 1.5 generally corresponded to initiation of the first failure criteria, corresponding to net section failure NS1. Failure due to bolt shear was the next predominate failure mode, followed by net section failure NS2, block shear, bearing, and net section failure NS3. The scaling factor at which each occurred varied greatly according to the earthquake ground motion under consideration. From the IDA results it can be seen that at a scaling factor of 6, most of the earthquake ground motions did not initiate the failure criteria of yield, corresponding to 2.5% storey drift.

Table 13 - Seven failure criteria in IDA analysis.

	Failure Criteria
i	NS 1 (3Bsa)
ii	NS 2 (3Cs)
iii	NS 3 (3As)
iv	Bolt Shear (D05X)
v	Block Shear (D06X)
vi	Bearing (D03X)
vii	Yield (drift)

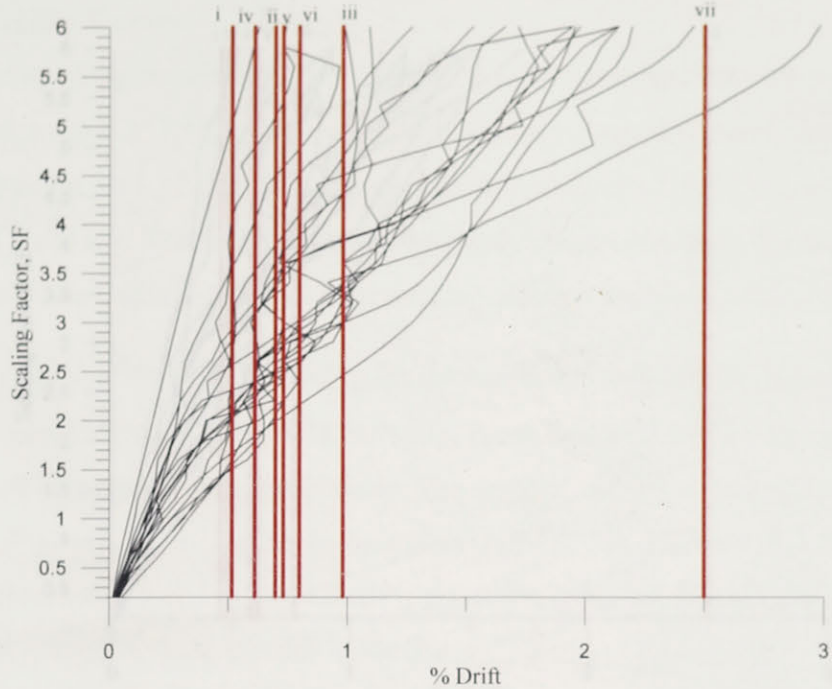

Figure 19 - Results of IDA analysis for building 2H.

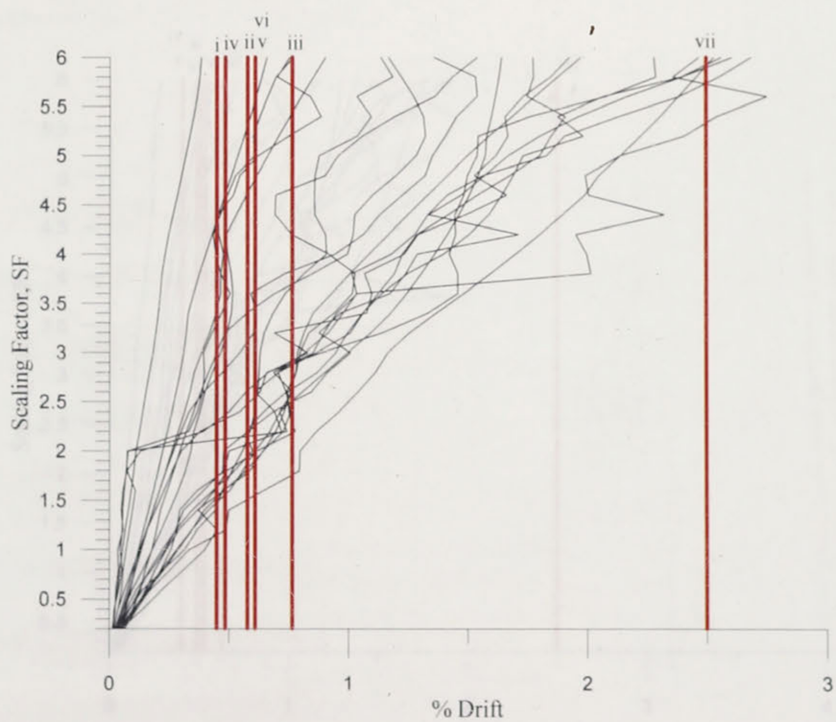

Figure 20 - Results of IDA analysis for building 7H.

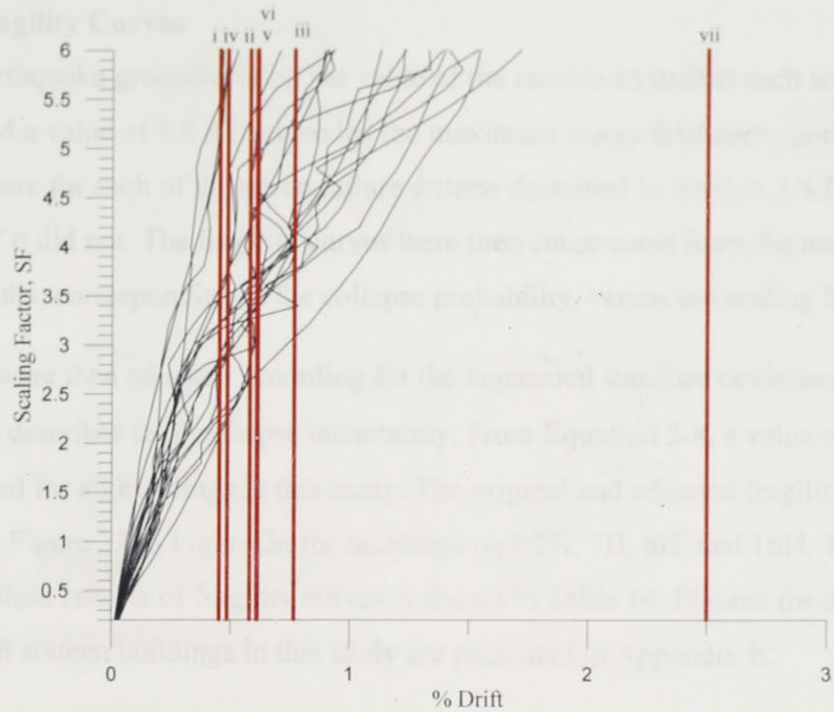

Figure 21 - Results of IDA analysis for building 8H.

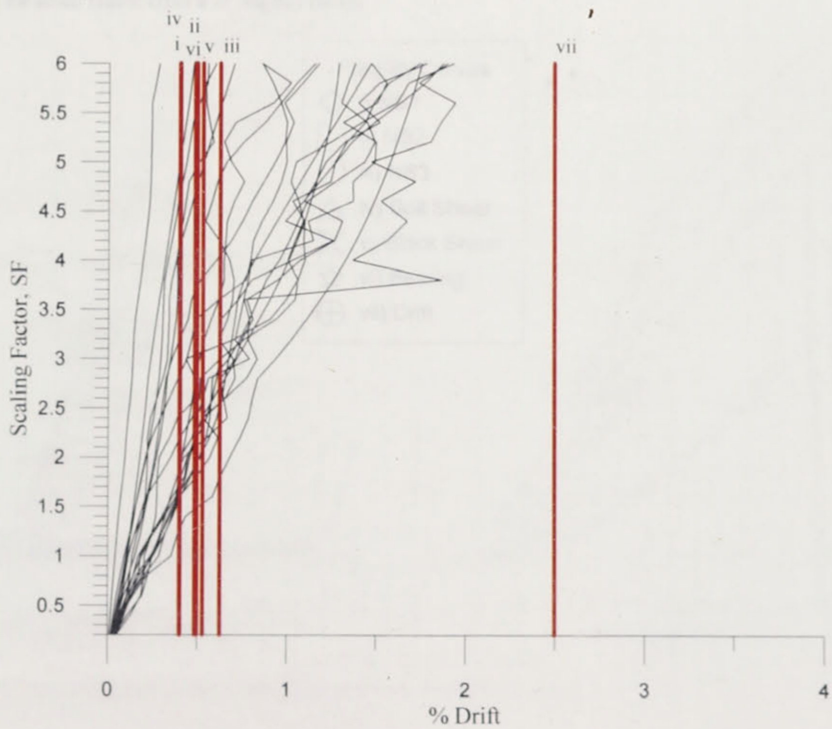

Figure 22 - Results of IDA analysis for building 16H.

4.2 Fragility Curves

For each earthquake ground motion the value of the maximum drift at each scaling factor was assigned a value of 1.0 if it exceeded the maximum storey drift corresponding to the onset of failure for each of the seven failure criteria described in Section 3.8.1, and a value of 0 if it did not. The fragility curves were then constructed from the median value of these results, corresponding to the collapse probability, versus the scaling factor.

The curves were then adjusted according for the lognormal standard deviation parameter, β_{TOT}, which describes total collapse uncertainty. From Equation 3-8, a value of $\beta_{TOT} = 0.80$ was used for all buildings in this study. The original and adjusted fragility curves are presented in Figure 23 to Figure 26 for buildings type 2H, 7H, 8H, and 16H. The legend for seven failure criteria of fragility curves is shown in Table 14. Figures for the fragility curves for all sixteen buildings in this study are presented in Appendix E.

Table 14 - Legend for seven failure criteria of fragility curves.

Fragility Curves	
◇	i) NS1
□	ii) NS2
○	iii) NS3
△	iv) Bolt Shear
×	v) Block Shear
☆	vi) Bearing
⊕	vii) Drift

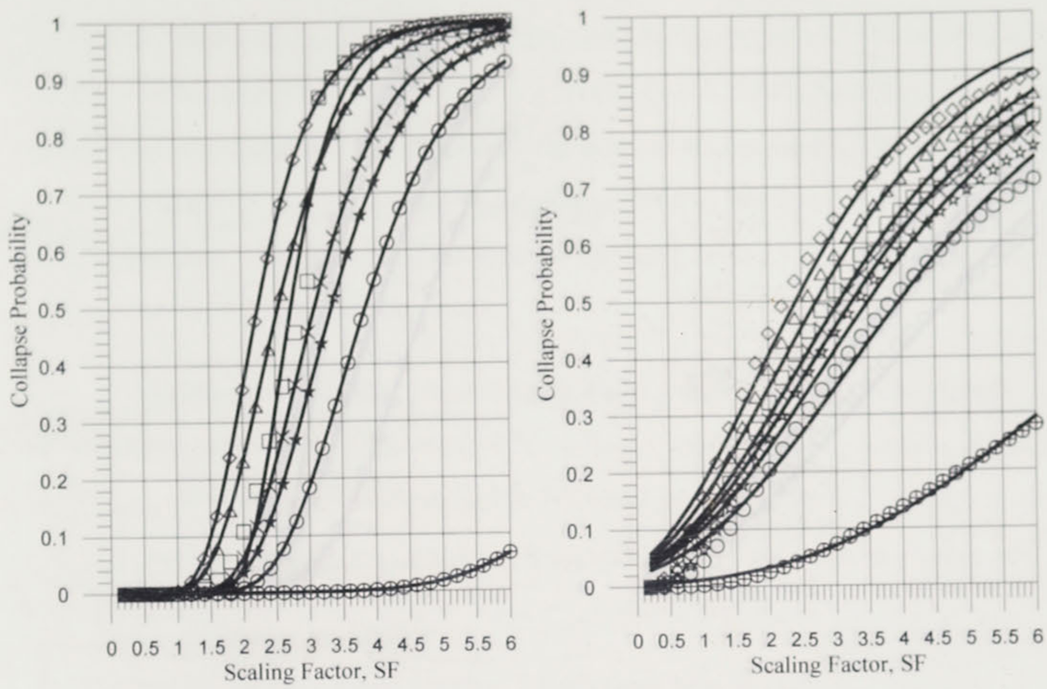

Figure 23 - Fragility curve (left), and adjusted curve for uncertainty (right) for building 2H.

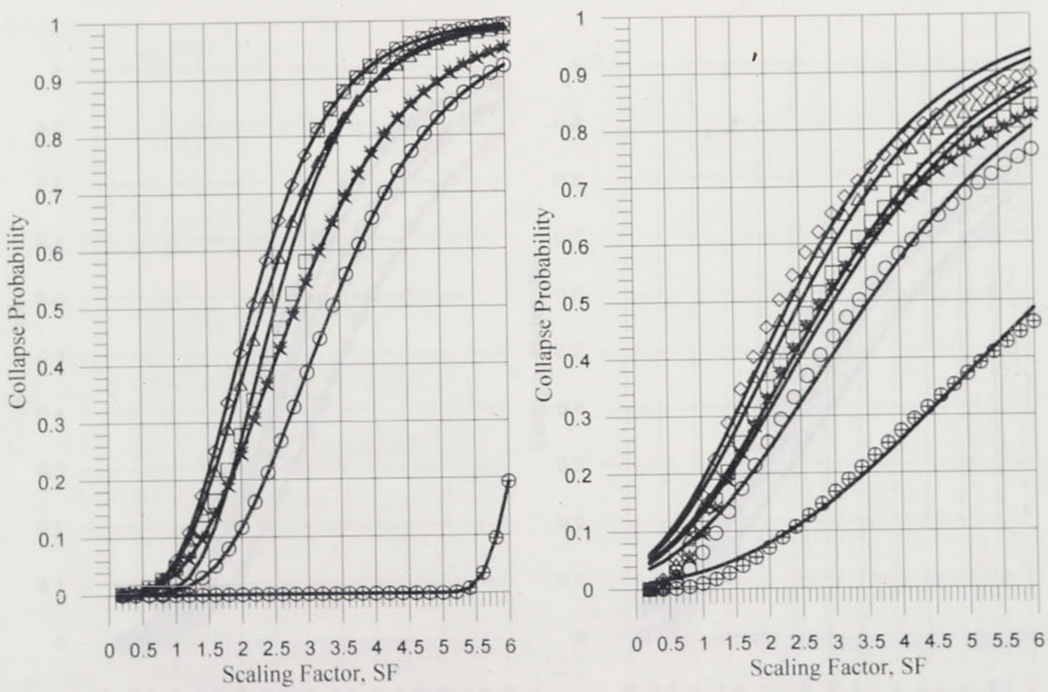

Figure 24 - Fragility curve (left), and adjusted curve for uncertainty (right) for building 7H.

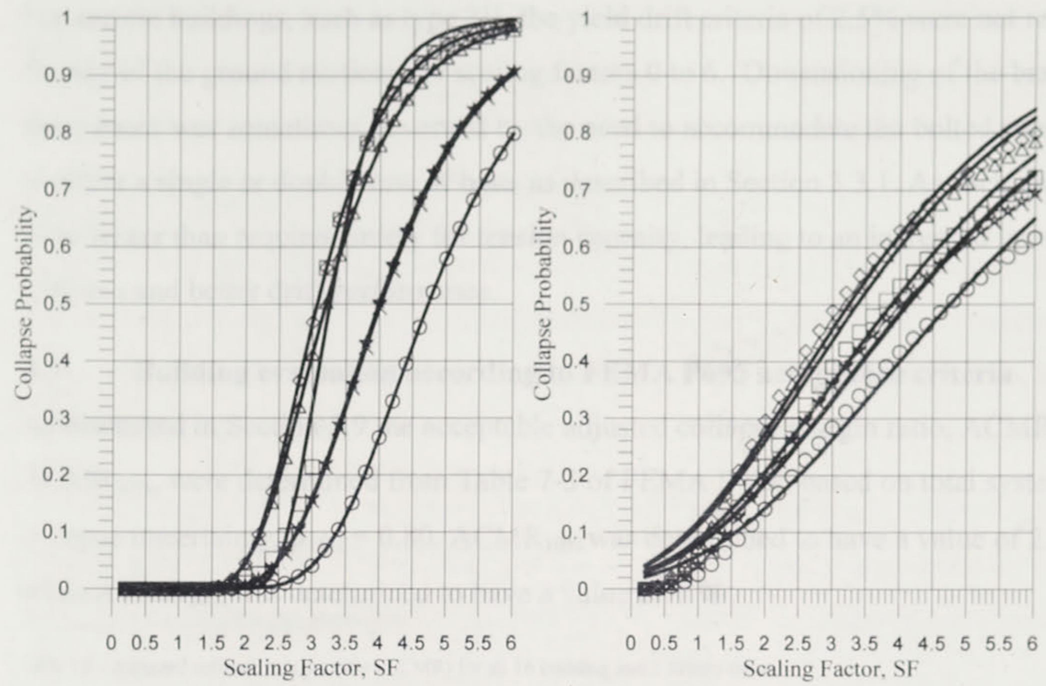

Figure 25 - Fragility curve (left), and adjusted curve for uncertainty (right) for building 8H.

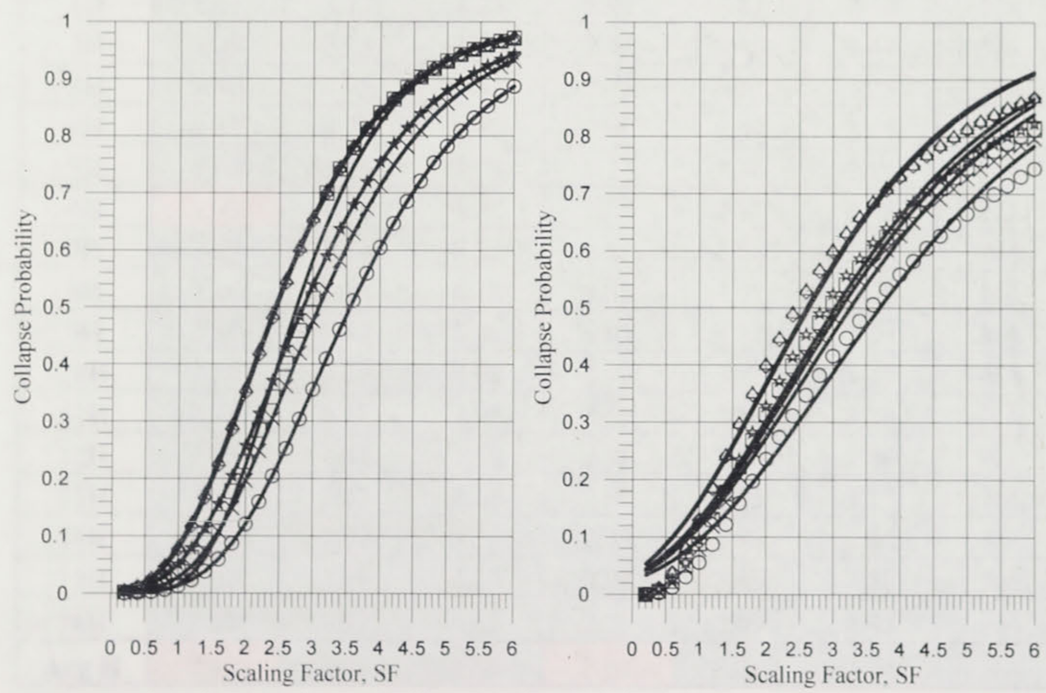

Figure 26 - Fragility curve (left), and adjusted curve for uncertainty (right) for building 16H.

For certain buildings, such as type 8H, the yield drift criteria of 2.5% were not reached for any of the ground motions for scaling factors 0 to 6. Dimensioning of the brace, in these cases was sometimes governed by the need to accommodate the bolted connection in either a single or double row of bolts as described in Section 3.3.1. As such, the braces were larger than required solely for tension capacity, leading to an increased lateral stiffness and better drift performance.

4.3 Building evaluation according to FEMA P695 acceptance criteria

As discussed in Section 3.9 the acceptable adjusted collapse margin ratio, $ACMR_{10\%}$ and $ACMR_{20\%}$, were determined from Table 7-3 of FEMA P-695 based on total system collapse uncertainty, $\beta_{TOT} = 0.80$. $ACMR_{10\%}$ was determined to have a value of 2.79, while $ACMR_{20\%}$ was determined to have a value of 1.96.

Table 15 - Adjusted collapse margin ratio (ACMR) for all 16 building and 7 failure criteria.

	ACMR Values						
Building	NS1	NS2	NS3	Bolt	Block	Bearing	Drift
1H	2.41	3.12	4.25	2.92	3.32	3.60	NA
2H	2.29	3.03	4.18	2.73	3.16	3.47	NA
3H	2.40	3.05	3.92	2.75	3.14	3.25	NA
4H	2.38	3.29	3.96	2.60	3.44	3.43	NA
5H	1.98	2.55	3.32	2.21	2.76	2.85	NA
6H	1.93	2.55	3.27	2.08	2.77	2.75	NA
7H	2.17	2.69	3.52	2.25	2.88	2.88	NA
8H	3.16	3.83	5.12	3.35	3.97	3.95	NA
9H	2.80	3.57	4.40	3.03	3.72	3.74	NA
10H	2.54	3.27	4.08	2.73	3.42	3.41	NA
11H	2.41	3.17	3.92	2.56	3.38	3.30	NA
12H	2.23	2.83	3.56	2.30	3.05	2.86	NA
13H	2.56	3.03	3.60	2.67	3.13	3.11	NA
14H	2.34	2.82	3.28	2.43	2.93	2.86	NA
15H	2.88	3.49	4.29	2.93	3.65	3.50	NA
16H	2.22	2.92	3.63	2.20	3.09	2.82	NA
Avg H	2.42	3.08	3.89	2.61	3.24	3.24	NA

For each of the seven failure criteria, the acceptable adjusted collapse margin ratio, $ACMR_{10\%}$, was compared to the average adjusted collapse margin ratio, ACMR, of all 16

buildings. Within each of the seven failure criteria, the acceptable adjusted collapse margin ratio was relaxed to a probability of collapse of 20%, $ACMR_{20\%}$, and compared to the adjusted collapse margin ratio, ACMR, of each of the 16 buildings individually. These results are presented in Table 15.

As shown in Table 15 acceptable performance was not achieved for NS1 (net section). In this case the average value of the adjusted collapse margin ratio and the individual value of the adjusted collapse margin ratio for building 6H did not exceed $ACMR10_{\%}$ and $ACMR20_{\%}$, respectively.

As well, acceptable performance was not achieved for the bolt shear failure criteria because although the individual values of the adjusted collapse margin ratio for all buildings exceeded $ACMR_{20\%} = 1.96$, the average value of the adjusted collapse margin ratio did not exceed $ACMR_{10\%} = 2.79$.

On average, the yield drift criteria of 2.5% were not reached for scaling factors 0 to 6. As such, a specific value of adjusted collapse margin ratio, ACMR, could not be calculated. However, as this limit was not reached it can be inferred that a failure criteria associated with drift was not critical for these buildings.

In general, although acceptable performance was not achieved in all cases, the buildings, on average, performed well.

4.4 Evaluation of an example building using dynamic modal analysis and NBCC 2010

Building type 7H was modelled using *Advance Design America* (ADA) software (Graitec 2010) using a dynamic modal analysis, as would be done by a design engineer evaluating an existing building using seismic design criteria as outlined in the NBCC 2010, and compared to the OpenSees results.

The seismic weight, W, was taken as the total weight of the structure as per NBCC 2010, and included the dead load and 25% of the design snow load.

The building as designed for the CSA S16-1965 *Steel Structures for Buildings* Standard (CSA 1965) and the 1965 National Building Code of Canada (NRCC 1965) was presented in Figure 11 and Figure 12.

The ductility-related force modification factor, R_d, was taken as 1.5 and the overstrength-related force modification factor, R_o, was taken as 1.3 for the case of conventional construction as defined by Clause 27.11 of CSA S16-2009. As such the elastic base shear, V_e, determined from dynamic analysis using the ADA model period was divided by the product of R_dR_o to determine the base shear from dynamic analysis, V_d. NBCC 2010 requires that the base shear from dynamic analysis, V_d, be scaled such that it is not less than 0.80 times the static design base shear, V, as presented in Section 3.4.3 of this report. The structure was considered regular as it did not present torsional sensitivity as defined by the NBCC 2010. As well, the static base shear, V, was determined with a ceiling on the fundamental period of $0.05h_n$ for braced frames, where h_n is the height of the structure.

As such the period was taken as 0.35 seconds for the building, and the spectral acceleration at the fundamental building period, S (T_a), was taken as 0.19. In comparison, the OpenSees and ADA model had periods of 0.78 sec and 1.26 sec respectively.

The calculation of final base shear is shown in Table 16. This value was controlled by the limit of 0.80 times the static design base shear, V.

Table 16 - Final base shear for analysis using base shear from dynamic analysis, V_d, and the design base shear, V.

V_e (kN)	V_d (kN)	V (kN)	0.8V (kN)	V_{final} (kN)
241	124	308	246	246

In the calculation of base shear the higher mode participation factor, M_v, and the importance factor for earthquake load for normal category buildings were both taken as 1.0.

From the ADA model the maximum tension force in each brace in the East-West bays was 102 kN. The braced frame had an elastic horizontal displacement of 9.87 mm in the East-West direction, corresponding to 0.0014% drift.

A category of conventional construction would, for Halifax, not entail additional verification that the diaphragm and connections of primary framing members were designed such that the failure mode was ductile, or for gravity loads combined with a seismic load multiplied by R_d =1.50 since $I_E F_a S_a(0.2) = 0.23$, which is less than the limit of 0.45 prescribed by Clause 27.11 of CSA S16-2009. However, due to the increase in seismic lateral load as compared to the 1965 design, the diaphragm fastener pattern presented in Table 5 was determined to be no longer adequate in the long direction (E-W).

4.4.1 Comparison example building using dynamic modal analysis and NBCC 2010 to OpenSees model results

For comparison to the dynamic modal analysis of building type 7H the results from the OpenSees model at scaling factor 1.0 were examined. This scaling factor corresponds to the UHS for Halifax as outlined in the NBCC 2010. The resulting brace tension force, T, base shear, V, and horizontal displacement, Δ_{brace}, are presented in Table 17 for the twenty earthquake ground motions studied.

Table 17 - Results of OpenSees analysis for ground motions 1 through 20.

	SF	T (kN)	V (kN)	Δ_{brace} (mm)
GM 1	1	63	80	8.35
GM 2	1	211	175	16.05
GM 3	1	115	111	11.37
GM 4	1	93	87	8.14
GM 5	1	208	173	15.26
GM 6	1	175	149	14.84
GM 7	1	96	89	9.15
GM 8	1	179	152	14.68
GM 9	1	210	185	20.62
GM 10	1	167	143	14.13
GM 11	1	33	41	3.68
GM 12	1	237	195	24.49
GM 13	1	176	153	15.28
GM 14	1	NA	NA	NA
GM 15	1	73	86	8.81
GM 16	1	211	179	21.95
GM 17	1	21	29	5.75
GM 18	1	41	50	8.68
GM 19	1	118	110	13.02
GM 20	1	23	36	9.20
AVG	1	123	111	12.17

The maximum tension force in each brace and horizontal displacement in the ADA model, which are calibrated to the requirements of 0.80 times the static base shear presented in the NBCC 2010, was 102 kN, with an elastic displacement of 9.87 mm.

From the OpenSees results for all ground motions, tension force and displacement varied greatly, however the average was 135 kN and 13.45 mm.

Tension resistance using CSA-S16-09 Clause 13.2 was calculated, from Equation 3-29 as 120 kN considering a bolted connection with 4 one inch bolts in a single row, where tension resistance calculations accounting for shear lag governed.

For the seven failure criteria identified in Section 3.8.1 of this report, the maximum drift corresponding to each is shown in Table 18.

Table 18 –Maximum drift for failure criteria of building 7H.

	NS1	NS2	NS3	Bolt	Block	Bearing	Yield
Max Drift (mm)	31.5	40.5	53.7	33.9	42.8	42.7	175.0
Max Drift (%)	0.45	0.58	0.77	0.48	0.61	0.61	2.50

Both the results for maximum horizontal displacement in the ADA and the OpenSees model did not surpass these limits. However, on average, the OpenSees results had higher brace forces and displacements than the ADA analysis using a ductility-related force modification factor and overstrength-related force modification factor for the case of conventional construction. This may imply that the force modification factors assumed in the case of conventional construction may not be applicable for the standard construction of all buildings since a nonlinear analysis may demonstrate higher design forces, although the elongation failure limits were not exceeded.

4.5 Summary

This report is complimentary to a study of similar scope by Caruso-Juliano (2012) evaluating the performance of CBFs in one-storey steel structures built with the 1965 National Building Code of Canada (NBCC) and CSA-S16-65 (CSA 1965) for the cities of Abbotsford and Montreal. Caruso-Juliano (2012) determined that for Abbotsford the performance was generally unsatisfactory for all seven failure criteria, while the same

buildings analysed for earthquake ground motions calibrated to the Montreal UHS performed satisfactorily. This is due to the fact that although seismic design criteria was the same for these two cities using the 1965 National Building Code of Canada (NRCC 1965), the uniform hazard spectrum specified in the 2010 National Building Code of Canada (NRCC 2010) varied greatly with Abbotsford having significantly higher spectral accelerations (Caruso-Juliano 2012).

For Halifax, in general, although acceptable performance was not achieved in all cases, the one-storey steel structures built with the 1965 National Building Code of Canada, on average, performed well, for the seven failure criteria outlined in this study.

In the NBCC 2010, a building's performance under seismic loading is primarily judged by the system satisfying strength criteria as defined in CSA S16-09 and drift limits. For buildings in the normal importance category a drift limit of 2.5% is specified in the NBCC. For all three cities (Abbotsford, Montreal, and Halifax) the 2.5% drift failure criteria was satisfied.

For Halifax, with a maximum scaling factor of 6, most of the earthquake ground motions did not initiate the failure criteria of yield, corresponding to 2.5% storey drift. For certain buildings, such as type 8H, the yield drift criteria of 2.5% were not reached for any of the ground motions for scaling factors 0 to 6. As such, a specific value of adjusted collapse margin ratio, ACMR, could not be calculated for the yield criteria. However, as this limit was not reached it can be inferred that a failure criteria associated with drift was not critical for these buildings. In some cases brace selection was often governed by the need to accommodate the bolted connection such that the braces were larger than required solely for tension capacity, leading to an increased lateral stiffness and better drift performance.

Chapter 5 – Conclusion

The behaviour of one-storey steel structures built with the 1965 National Building Code of Canada (NRCC 1965) and CSA-S16-65 (CSA 1965) under current building code standards for seismic design was studied in order to evaluate the performance of CBFs in order to provide recommendations for seismic evaluation and rehabilitation requirements for such existing buildings for future building codes. This was done for the city of Halifax as a complimentary study to one of a similar scope by Caruso-Juliano (2012) evaluating the performance of CBFs in one-storey steel structures built with the 1965 National Building Code of Canada (NBCC) and CSA-S16-65 (CSA 1965) for the cities of Abbotsford and Montreal.

The response of a series of sixteen one-storey buildings with varying aspect ratios and heights was studied, subjected to ten artificial and ten historical earthquake ground motions, using an analytical OpenSees, *Open System for Earthquake Engineering Simulation* (OpenSees 2011) model for nonlinear time history dynamic analysis.

5.1 Analysis Conclusions

The intended performance level in the design earthquakes, as well as the acceptance criteria used in the braced frame analysis was established using FEMA P695 (FEMA 2009) criteria.

Although acceptable performance was not achieved in all cases, the one-storey steel structures built with the 1965 National Building Code of Canada, on average, performed well, for the seven failure criteria outlined in this study for the city of Halifax.

In comparison, Caruso-Juliano (2012) determined that for Abbotsford the performance was generally unsatisfactory for all seven failure criteria, while the same buildings analysed for earthquake ground motions calibrated to the Montreal UHS performed satisfactorily, since seismic design criteria was the same for these two cities using the 1965 National Building Code of Canada (NRCC 1965).

An example building was compared using a dynamic modal analysis, as would be done by a design engineer evaluating an existing building using seismic design criteria as outlined in the NBCC 2010, to the non linear OpenSees results. On average, the

OpenSees model results had higher brace forces and displacements than the dynamic modal analysis analysis using a ductility-related force modification factor and overstrength-related force modification factor for the case of conventional construction. This may imply that the force modification factors assumed in the case of conventional construction may not be applicable for the construction of all buildings since a nonlinear analysis may demonstrate higher design forces. The elongation failure limits, as defined for the seven failure criteria, however, were not exceeded for either case.

5.2 Recommendations for future research

To account for failure modes not explicitly modeled using OpenSees failure criteria associated with brace elongation was used to account for connection failure in the performance evaluation process. Modes of failure considered were net section failure, bearing failure, bolt shear, and block shear failure. Additional testing of existing brace specimens could be conducted in order to include additional model degradation mechanisms in the modelling and supplement the seven modes of failure seen in this study.

As well, considering base shears calculated with the 2010 National Building Code of Canada exceed those calculated using the 1965 code, diaphragm strengthening may be required for such existing buildings. As suggested by Caruso-Juliano (2012), studying the sixteen one-storey buildings in this study with a diaphragm retrofit would be valuable for comparison to the unreinforced case.

Additionally, it was seen that the force and deformation of an example building validated using dynamic modal analysis as outlined in the NBCC 2010, and assuming "conventional construction" may be less than that using a nonlinear analysis. This may imply that the force modification factors assumed in the case of conventional construction may not be applicable to all building cases. Only one building was studied for this case, and for such a conclusion to be drawn, however, additional research should be undertaken, including analysis of other building sites besides Halifax.

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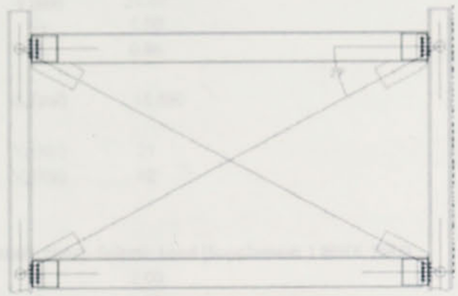
Appendix A

Design of building 7H for 1965 National Building Code of Canada (NRCC 1965) and CSA-S16-65 (CSA 1965).

	CALCULATION SHEET	PROJECT NO.: M.ENG
CLIENT: NA	PREPARED: A.G.	PAGE 1 OF 2
PROJECT: 7H - 30MX60M	CODE: CSA S16-1965	
SUBJECT: Basic Loads -1965	DATE:	

Location: **HALIFAX**



Frame Height	7000
Building Height	7000
Bay Width (mm)	7500
θ	43.03
Brace Length	10259
w (m)	30.0
l (m)	60.0

Trib Area	56.25	int column
Trib Area	28.13	ext column

Dead Load

4 Ply Ashphalt + Gravel	0.32
100 mm Rigid Foam	0.03
12.5 mm Gypsum	0.10
0.91 mm Steel Deck	0.10
Ductwork	0.25
Fire Protection	0.07
Joists	0.10
Beams	0.15
Total (kPa)	1.12
Total (psf)	23.33

Assumption

Exterior Walls (kPa)	1.50
Exterior Walls (psf)	31.25

Snow Load

S (psf) = 36.04 S (kPa) = 1.73

	CALCULATION SHEET	PROJECT NO.: M.ENG
CLIENT: NA	PREPARE A.G.	PAGE OF
PROJECT: 30MX60M	CODE: CSA S16-1965	2 2
SUBJECT: Basic Loads -1965	DATE:	

Climatic Data - Wind Load (Supplement 1 NBCC 1965)

v (mph)	90.0			
q (psf)	21.87			
C_h	1.00			
C_p	0.85			
q_w (psf)	18.590		q_w (kPa)	0.892
V_x (kip)	21		V_x (kN)	94
V_z (kip)	42		V_z (kN)	187

Climatic Data - Seismic Load (Supplement 1 NBCC 1965)

R	2.00			
C	1.25			
I	1.00			
F	1.00			
S	0.025			
K	0.0625			
$V_{x/z}$ (kip)	42		$V_{x/z}$ (kN)	185

1 - storey :	h_n (m) =	7
Area :	A (m^2) =	1800
Perimeter:	P (m) =	180

W_1 (kip) = 666 Single-Storey

W_1 (kN) = 2961 Single-Storey

$V_{x/brace}$ (kN)	$V_{x/brace}$ (kip)
93.7	21.1

WIND GOVERNS

$T_{f/brace}$ (kN)	$T_{f/brace}$ (kip)
128.2	28.8

	CALCULATION SHEET	PROJECT NO.: M.ENG
CLIENT: NA	PREPARED: A.G.	PAGE 1 OF 1
PROJECT: 7H - 30MX60M	CODE: CSA S16-1965	
SUBJECT: Tension only bracing -1965	DATE:	

F_y	44	ksi	F_u	65	ksi
E	29000	ksi			

BRACING - ROOF

Single Angle: L 2.5 x 2.5 x 1/4

L	404 in			
A	1.19	in^2	k_x	0.5
t	0.25	in	k_y	0.5
r_x	0.77	in		
r_y	0.77	in		

Connection (A307 bolts): 1 in

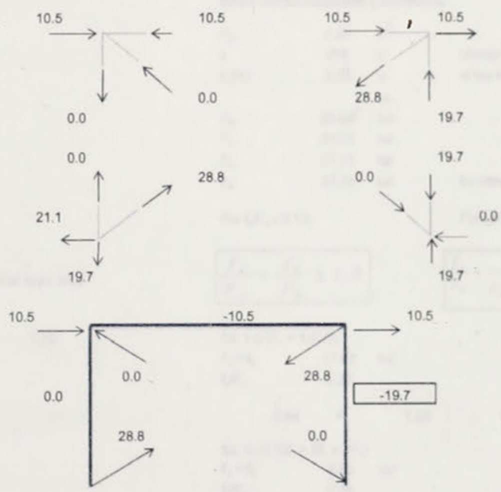
F_t	14	ksi	A_b	0.785	in^2
F_v	10	ksi	d_b	1.0625	in

V_r (kip/bolt) **7.85**

Level	$(kL/r)_x$	$(kL/r)_y$	T_f (kip)	# rows bolts	# bolts req'd / row	A_n (in^2)	T_r (kip)	
Roof	262	262	29	1	4	0.92	33	0.89
	OK	OK						

Top Force	21.1
Per Side	10.5
θ	43.03

δ	0.337	in
Δ	0.247	in



	CALCULATION SHEET	PROJECT NO.: M.ENG
CLIENT: NA	PREPARED: A.G.	PAGE 1 OF 1
PROJECT: 7H - 30MX60M	CODE: CSA S16-1965	
SUBJECT: Column at bracing -1965	DATE:	

F_y	44 ksi				
E	29000 ksi				
A_T (ft²)	303 (for exterior column)		q_w (psf)	18.59	
DL =	7.06 kip		t_w (ft)	24.61	Tributary width wind load
SL =	10.91 kip		H (ft)	22.97	
WL =	19.66 kip		w_w (lb/ft)	457.42	
EL =	19.41 kip				

1. 1.0 DL + 1.0 SL	C_f =	17.97 kip		
2a. 1.0 DL + 1.0 WL	C_f =	26.72 kip	$M_{w,x}$ (kip-ft)	30.16
2b. 1.0 DL + 1.0 EL	C_f =	30.33 kip		
3a. 0.75 (DL + SL + WL)	C_f =	28.22 kip	$M_{w,x}$ (kip-ft)	22.62
3b. 0.75 (DL + SL + EL)	C_f =	37.49 kip		

Column Selection: 8WF35 — Compact section

A	10.30	in²			
b	8.03	in	t	0.493	in
d	8.12	in	w	0.315	in
I_x	126	in⁴	I_y	42.5	in⁴
S_x	31.1	in³	S_y	10.6	in³
r_x	3.5	in	r_y	2.03	in
Z_x	34.7	in³			

AXIAL COMPRESSION:

k_x	1.0	
k_y	1.0	
$(kL/r)_x$	79	OK
$(kL/r)_y$	136	OK
C_a	20.0	
C_p	96.09	
m	0.135	
F_a	8.08	ksi

3b. 0.75 (DL + SL + EL) is worst case for axial load only

f_a	3.64 ksi		
f_a/F_a	**0.450**	**<**	**1.00**
	OK		

AXIAL COMPRESSION + BENDING:

A_{tc}	3.96	in²	
L	276	in	Unsuported length
r_t (in)	2.22	in	of tee section
F_{bt}	29.04	ksi	
F_{bc}	21.23	ksi	
F_b	21.23	ksi	
F_e'	24.03	ksi	Bending in x-axis

For $f_a/F_a \le 0.15$:

$$\boxed{\frac{f_a}{F_a} + \frac{f_b'}{F_b} \le 1.0}$$

For $f_a/F_a > 0.15$:

$$\boxed{\frac{f_a}{F_a} + \frac{f_b'}{F_b\left(f_a/F_e'\right)} \le 1.0}$$

2a. 1.0 DL + 1.0 WL

$f'_b = f_b$	11.64	ksi
f_a/F_a	0.32	

0.94 < 1.00 OK

3a. 0.75 (DL + SL + WL)

$f'_b = f_b$	8.73	ksi
f_a/F_a	0.34	

0.80 < 1.00 OK

	CALCULATION SHEET	PROJECT NO.: M.ENG
CLIENT: NA	PREPARED: A.G.	PAGE OF
PROJECT: 7H - 30MX60M	CODE: CSA S16-1965	1 1
SUBJECT: EXTERIOR Column -1965	DATE:	

F_y	44	ksi
E	29000	ksi

A_T (ft^2)	303 (for exterior column)	q_w (psf)	18.59
DL =	7.06 kip	t_w (ft)	24.61 Tributary width wind load
SL =	10.91 kip	H (ft)	22.97
WL =	kip	w_w (lb/ft)	457.42
EL =	kip		

1. 1.0 DL + 1.0 SL	C_f =	17.97 kip		
2a. 1.0 DL + 1.0 WL	C_f =	7.06 kip	$M_{w,x}$ (kip-ft)	30.16
2b. 1.0 DL + 1.0 EL	C_f =	10.91 kip		
3a. 0.75 (DL + SL + WL)	C_f =	13.48 kip	$M_{w,x}$ (kip-ft)	22.62
3b. 0.75 (DL + SL + EL)	C_f =	8.18 kip		

Column Selection: 8WF31 Compact section

A	9.12	in^2			
b	8.00	in	t	0.433	in
d	8.00	in	w	0.288	in
I_x	110	in^4	I_y	37.0	in^4
S_x	27.4	in^3	S_y	9.2	in^3
r_x	3.47	in	r_y	2.01	in
Z_x	30.4	in^3			

AXIAL COMPRESSION:

k_x	1.0	
k_y	1.0	
$(kL/r)_x$	79	OK
$(kL/r)_y$	137	OK
C_o	20.0	
C_p	96.09	
m	0.135	
F_a	7.93	ksi

1. 1.0 DL + 1.0 SL **is worst case for axial load only**

f_a 1.97 ksi

f_a/F_a **0.249 < 1.00**

OK

AXIAL COMPRESSION + BENDING:

A_{tc}	3.46	in^2	
L	276	in	Unsuported length
r_t (in)	2.20	in	of tee section
F_{bt}	29.04	ksi	
F_{bo}	18.85	ksi	
F_b	18.85	ksi	
F_e'	23.62	ksi	Bending in x-axis

For $f_a/F_a \le 0.15$:

$$\boxed{\frac{f_a}{F_a} + \frac{f_b'}{F_b} \le 1.0}$$

For $f_a/F_a > 0.15$:

$$\boxed{\frac{f_a}{F_a} + \frac{f_b'}{F_b\left(f_a/F_e'\right)} \le 1.0}$$

2a. 1.0 DL + 1.0 WL

$f_b' = f_b$ 13.21 ksi

f_a/F_a 0.10

0.80 < 1.00 OK

3a. 0.75 (DL + SL + WL)

$f_b' = f_b$ 9.91 ksi

f_a/F_a 0.19

0.75 < 1.00 OK

	CALCULATION SHEET	PROJECT NO.: M.ENG
CLIENT: NA	PREPARED: A.G.	PAGE OF
PROJECT: 7H - 30MX60M	CODE: CSA S16-1965	1 1
SUBJECT: INTERIOR Column -1965	DATE:	

F_y	50	ksi
E	29000	ksi

A_T (ft^2)	605 (for interior column)	q_w (psf)	18.59
DL =	14.13 kip	t_w (ft)	0.00 Tributary width wind load
SL =	21.82 kip	H (ft)	22.97
WL =	kip	w_w (lb/ft)	0.00
EL =	kip		

1. 1.0 DL + 1.0 SL	C_f =	35.95 kip		
2a. 1.0 DL + 1.0 WL	C_f =	14.13 kip	$M_{w,x}$ (kip-ft)	0.00
2b. 1.0 DL + 1.0 EL	C_f =	21.82 kip		
3a. 0.75 (DL + SL + WL)	C_f =	26.96 kip	$M_{w,x}$ (kip-ft)	0.00
3b. 0.75 (DL + SL + EL)	C_f =	16.37 kip		

Column Selection: HSS 6X6X0.188 — Non-compact section

A	4.28	in^2
b	6.00	in
d	6.00	in
t	0.188	in
r_x	2.36	in
r_y	2.36	in

AXIAL COMPRESSION:

k_x	1.0	
k_y	1.0	
$(kL/r)_x$	116.78	OK
$(kL/r)_y$	116.78	OK
C_o	20.0	
C_p	87.95	
m	0.158	
F_a	10.93	ksi

1. 1.0 DL + 1.0 SL **is worst case for axial load only**

f_a 8.40 ksi

f_a/F_a **0.769** < **1.00** OK

	CALCULATION SHEET	PROJECT NO.: M.ENG
CLIENT: NA	PREPARED: A.G.	PAGE 1 OF 1
PROJECT: 7H - 30MX60M	CODE: CSA S16-1965	
SUBJECT: EXT Beam -1965	DATE:	

F_y	44	ksi
E	29000	ksi

t_w(ft)	12.30	
DL =	287.07	lb/ft
SL =	443.43	lb/ft

1. 1.0 DL + 1.0 SL w_f = 730 lb/ft

L (ft)	24.61		
M_x(kip-ft)	55.29		
I_x >	102.51	in^4	FOR $\Delta \leq L/240$
Lu (ft)	6.15		

Beam Selection: 12B22 Compact section

A	6.47	in^2				
b	4.03	in		t	0.424	in
d	12.31	in		w	0.260	in
I_x	156	in^4	OK	I_y	3.67	in^4
S_x	25.3	in^3		S_y	1.83	in^3
r_x	4.91	in		r_y	0.81	in
Z_x	24.8	in^3				

BENDING:

A_{fc}	1.7087	in^2	
L	0	in	Unsuported length
r_t (in)	1.02	in	of tee section

F_{bt}	29.04	ksi
F_{bc}	29.04	ksi
F_b	29.04	ksi
$f'_b = f_b$	26.22	ksi

f'_b/F_b **0.90 < 1.00 OK**

	CALCULATION SHEET	PROJECT NO.: M.ENG
CLIENT: NA	PREPARED: A.G.	PAGE OF
PROJECT: 7H - 30MX60M	CODE: CSA S16-1965	1 3
SUBJECT: INT GERBER Beam -1965	DATE:	

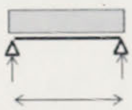
F_y	44	ksi
E	29000	ksi

L_w(ft) 24.61
DL = 574.15 lb/ft
SL = 886.85 lb/ft

1. 1.0 DL + 1.0 SL w_f = 1461 lb/ft
2. 1.0 DL + 0.50 SL w_f = 1018 lb/ft

BEAM - SIMPLY SUPPORTED:

L c/c (ft)	24.61		
Gerber ext. (ft)	3.61		
L (ft)	17.39		
M_x (kip-ft)	55.22		
R_x (kip)	12.70		
I_x >	72.35	in^4	FOR $\Delta \leq L/240$
Lu (ft)	6.15		



Beam Selection: 12B22 Compact section

A	6.47	in^2				
b	4.03	in		t	0.424	in
d	12.31	in		w	0.260	in
I_x	156	in^4	OK	I_y	3.67	in^4
S_x	25.3	in^3		S_y	1.83	in^3
r_x	4.91	in		r_y	0.81	in
Z_x	24.8	in^3				

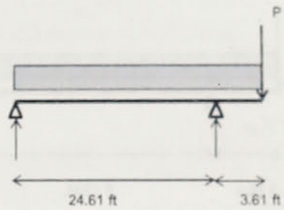
BENDING:

A_{fc}	1.7087	in^2	
L	0	in	Unsuported length
r_t (in)	1.02	in	of tee section
F_{bt}	29.04	ksi	
F_{bc}	29.04	ksi	
F_b	29.04	ksi	
$f'_b = f_b$	26.19	ksi	

f'_b/F_b **0.90** < **1.00** OK

	CALCULATION SHEET	PROJECT NO.: M.ENG
CLIENT: NA	PREPARED: A.G.	PAGE 2 OF 3
PROJECT: 30MX60M	CODE: CSA S16-1965	
SUBJECT: INT GERBER Beam -1965	DATE:	

CANTILIVER BEAM 1:



L c/c (ft)	24.61
Gerber ext. (ft)	3.61

Load case (1) - 1.0 DL + 1.0 SL

P (kip)	12.70
$M_{x(+)}$(kip-ft)	82.90
$M_{x(-)}$(kip-ft)	55.36

Load case (2a) - 1.0 DL + 1.0 SL on cantiliver and 1.0 DL +0.50 SL on center span

P (kip)	12.70
$M_{x(+)}$(kip-ft)	49.34
$M_{x(-)}$(kip-ft)	55.36

Load case (2b) - 1.0 DL + 0.50 SL on cantiliver and 1.0 DL +1.0 SL on center span

P (kip)	8.85
$M_{x(+)}$(kip-ft)	91.30
$M_{x(-)}$(kip-ft)	38.55

Beam Selection: 14WF30 Compact section

A	8.81	in^2			
b	6.73	in	t	0.383	in
d	13.86	in	w	0.270	in
I_x	290	in^4	I_y	17.5	in^4
S_x	41.8	in^3	S_y	5.2	in^3
r_x	5.73	in	r_y	1.41	in
Z_x	47.1	in^3			

BENDING:

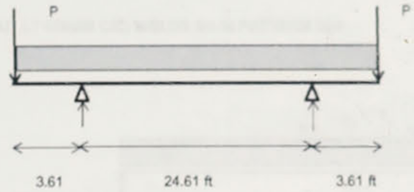
A_{tc}	2.5776	in^2	
r_t (in)	1.75	in	of tee section

(a) Negative moment					(b) Postive moment				
L	43	in	Unsuported length		L	0	in	Unsuported length	
F_{bt}	29.04	ksi			F_{bt}	29.04	ksi		
F_{bc}	29.04	ksi			F_{bc}	29.04	ksi		
F_b	29.04	ksi			F_b	29.04	ksi		
$f'_b = f_b$	15.89	ksi			$f'_b = f_b$	26.21	ksi		
f'_b/F_b	**0.55**	**<**	**1.00**	**OK**	f'_b/F_b	**0.90**	**<**	**1.00**	**OK**

	CALCULATION SHEET	PROJECT NO.: M.ENG
CLIENT: NA	PREPARED: A.G.	PAGE OF
PROJECT: 30MX60M	CODE: CSA S16-1965	3 3
SUBJECT: INT GERBER Beam -1965	DATE:	

CANTILIVER BEAM 2:

L c/c (ft)	24.61
Gerber ext. (ft)	3.61



Load case (1) - 1.0 DL + 1.0 SL

P (kip)	12.70
$M_{x(+)}$(kip-ft)	55.22
$M_{x(-)}$(kip-ft)	55.36

Load case (2a) - 1.0 DL + 1.0 SL on cantiliver and 1.0 DL +0.50 SL on center span

P (kip)	12.70
$M_{x(+)}$(kip-ft)	21.66
$M_{x(-)}$(kip-ft)	55.36

Load case (2b) - 1.0 DL + 0.50 SL on cantiliver and 1.0 DL +1.0 SL on center span

P (kip)	8.85
$M_{x(+)}$(kip-ft)	72.02
$M_{x(-)}$(kip-ft)	38.55

Beam Selection: 14B26   Compact section

A	7.65	in^2			
b	5.02	in	t	0.418	in
d	13.89	in	w	0.255	in
I_x	243	in^4	I_y	8.26	in^4
S_x	34.9	in^3	S_y	3.29	in^3
r_x	5.63	in	r_y	1.04	in
Z_x	39.8	in^3			

BENDING:

A_{fc}	2.0984	in^2	
r_t (in)	1.29	in	of tee section

(a) Negative moment				(b) Postive moment			
L	43	in	Unsuported length	L	0	in	Unsuported length
F_{bx}	29.04	ksi		F_{bx}	29.04	ksi	
F_{bc}	29.04	ksi		F_{bc}	29.04	ksi	
F_b	29.04	ksi		F_b	29.04	ksi	
$f'_b = f_b$	19.03	ksi		$f'_b = f_b$	24.76	ksi	

f'_b/F_b	0.66	<	1.00	OK	f'_b/F_b	0.85	<	1.00	OK

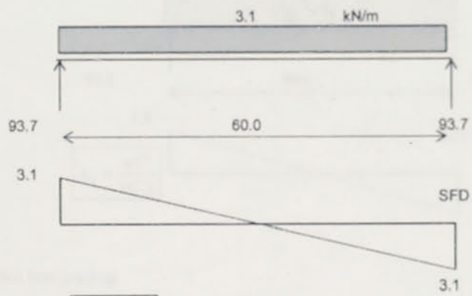
	CALCULATION SHEET	PROJECT NO.: M.ENG
CLIENT: NA	PREPARED: A.G.	PAGE OF
PROJECT: 7H - 30MX60M	CODE: CSA S16-1965	1 2
SUBJECT: DIAP -1965	DATE:	

DECK : **22 GAGE - 38 MM - SIDELAP AT 600MM C/C; WELDS 3/4 IN PATTERN 36/4**

q_r **3.95 kN/m**

G' **2.75 kN/mm**

L **1875 mm**



Long Direction:

V_x (kN)	187.4
M_{max} (kNm)	1405
F_{rave} (kN)	47

v_f = 3.1 kN/m

OK

$$\boxed{\Delta_B = \frac{PL}{AE}\cos\theta} \quad \boxed{\Delta_F = \frac{5wL^4}{384EI}} \quad \boxed{\Delta_S = \frac{wL^2}{8G'b}}$$

I 1.88 x10^{12} mm^4

Δ_B 6.26 mm (Deflection from bracing)

Δ_F 1.40 mm

Δ_s 17.03 mm

Δ_B **6.26 mm** K_B **29.93 kN/mm**

Δ_D **18.44 mm** K_D **13.57 kN/mm**

$\Delta_{elastic}$ **24.70 mm**

From ASCE-41 for flexible diaphragms:

$$\boxed{T \approx \sqrt{\frac{W}{V}\left(0.004\Delta_B + 0.0031\Delta_D\right)}}$$

T = 1.14 s

From Medhekar:

$$\boxed{T \approx 2\pi\sqrt{\frac{(K_B + K_D)W}{gK_BK_D}}}$$

$T_{medhekar}$ = 1.13 s

From NBCC 2010:

$$\boxed{T \approx 0.025h_n}$$

T_{NBCC} = 0.18 s

= 0.35 s if verified by dynamic analysis

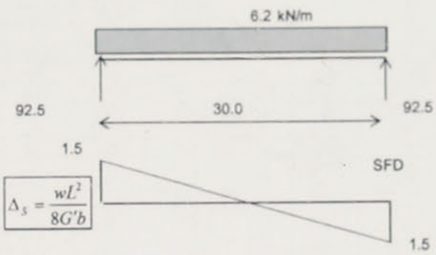
	CALCULATION SHEET	PROJECT NO.: M.ENG
CLIENT: NA	PREPARED: A.G.	PAGE 2 OF 2
PROJECT: 7H - 30MX60M	CODE: CSA S16-1965	
SUBJECT: DIAP -1965	DATE:	

Short Direction:

V_s (kN)	185.1
M_{max} (kNm)	694
F_{cave} (kN)	12

v_f = 1.5 kN/m

OK



$\boxed{\Delta_B = \dfrac{PL}{AE}\cos\theta}$ $\boxed{\Delta_F = \dfrac{5wL^4}{384EI}}$ $\boxed{\Delta_S = \dfrac{wL^2}{8G'b}}$

I 7.51 $\times 10^{12}$ mm^4

Δ_B	6.26 mm	(Deflection from bracing)
Δ_F	0.04 mm	
Δ_s	4.21 mm	

Δ_B	**6.26 mm**	K_B	**29.56 kN/mm**
Δ_D	**4.25 mm**	K_D	**54.28 kN/mm**

$\Delta_{elastic}$ **10.51 mm**

From ASCE-41 for flexible diaphragms:

$$\boxed{T \approx \sqrt{\frac{W}{V}\left(0.004\Delta_B + 0.0031\Delta_D\right)}}$$

T = 0.78 s

From Medhekar:

$$\boxed{T \approx 2\pi\sqrt{\frac{(K_B + K_D)W}{gK_BK_D}}}$$

$T_{medhekar}$ = 0.79 s

From NBCC 2010:

$$\boxed{T \approx 0.025h_n}$$

T_{NBCC} = 0.18 s
= 0.35 s if verified by dynamic analysis

SUMMARY PERIODS:

	ASCE-41 for flexible diaphragms $T \approx \sqrt{\frac{W}{V}\left(0.004\Delta_B + 0.0031\Delta_D\right)}$	Medhekar $T \approx 2\pi\sqrt{\frac{(K_B + K_D)W}{gK_BK_D}}$	NBCC 2010 (*) $T \approx 0.025h_n$
Long direction:	1.14	1.13	0.18
Short direction:	0.78	0.79	0.18

(*) Can be multiplied by 2 if dynamic analysis is performed.

Appendix B

IDA analysis and results of building 7H from OpenSees data.

	CALCULATION SHEET	PROJECT NO.: M.ENG
CLIENT: MCGILL	PREPARED: A.G.	PAGE OF
PROJECT: CBFs	CODE: NA	1 1
SUBJECT: Failure Criteria based on testing	DATE: 2012-12-01	

Failure	δ_u (mm)	P (kN)	L_{tot} (mm)	L_{core} (mm)	A_{sect} (mm^2)	δ_u - PL/EA
NS 1 (3Bsa)	12.1	402.0	3070	228.6	1362	7.91
NS 2 (3Cs)	18.1	256.0	2880	228.6	845	14.08
NS 3 (3As)	28.1	258.6	2450	228.6	929	25.01
Bearing (D03X)	22.7	338.3	856	-	1858	21.92
Bolt Shear (D05X)	15.7	363.7	916	-	1858	14.80
Block Shear (D06X)	15.5	621.0	796	-	1858	14.17

			NS 1 (3Bsa)			NS 2 (3Cs)			NS 3 (3As)		
Structure Type	L_{brace} (mm)	A_{brace} (mm^2)	δ_u + PL/EA	%	Elong	δ_u + PL/EA	%	Elong	δ_u + PL/EA	%	Elong
1	7315	355	18.7	0.26	**1.0026**	25.2	0.34	**1.0034**	35.2	0.48	**1.0048**
2	7071	355	18.3	0.26	**1.0026**	24.8	0.35	**1.0035**	34.8	0.49	**1.0049**
3	8327	394	20.2	0.24	**1.0024**	26.7	0.32	**1.0032**	36.6	0.44	**1.0044**
4	10440	523	23.3	0.22	**1.0022**	29.9	0.29	**1.0029**	39.5	0.38	**1.0038**
5	8660	606	20.7	0.24	**1.0024**	27.2	0.31	**1.0031**	37.1	0.43	**1.0043**
6	10535	581	23.5	0.22	**1.0022**	30.0	0.29	**1.0029**	39.7	0.38	**1.0038**
7	10259	768	23.0	0.22	**1.0022**	29.6	0.29	**1.0029**	39.3	0.38	**1.0038**
8	10440	1490	23.3	0.22	**1.0022**	29.9	0.29	**1.0029**	39.5	0.38	**1.0038**
9	9861	1490	22.5	0.23	**1.0023**	29.0	0.29	**1.0029**	38.7	0.39	**1.0039**
10	10226	1490	23.0	0.22	**1.0022**	29.6	0.29	**1.0029**	39.2	0.38	**1.0038**
11	11136	1490	24.3	0.22	**1.0022**	31.0	0.28	**1.0028**	40.5	0.36	**1.0036**
12	12490	1490	26.3	0.21	**1.0021**	33.0	0.26	**1.0026**	42.4	0.34	**1.0034**
13	10706	1490	23.7	0.22	**1.0022**	30.3	0.28	**1.0028**	39.9	0.37	**1.0037**
14	11907	1490	25.5	0.21	**1.0021**	32.1	0.27	**1.0027**	41.6	0.35	**1.0035**
15	12845	1852	26.9	0.21	**1.0021**	33.5	0.26	**1.0026**	42.9	0.33	**1.0033**
16	14318	1852	29.0	0.20	**1.0020**	35.8	0.25	**1.0025**	44.9	0.31	**1.0031**

			Bearing (D03X)			Bolt Shear (D05X)			Block Shear (D06X)		
Structure Type	L_{brace} (mm)	A_{brace} (mm^2)	δ_u + PL/EA	%	Elong	δ_u + PL/EA	%	Elong	δ_u + PL/EA	%	Elong
1	7315	355	28.6	0.39	**1.0039**	22.0	0.30	**1.0030**	26.4	0.36	**1.0036**
2	7071	355	28.4	0.40	**1.0040**	21.7	0.31	**1.0031**	26.0	0.37	**1.0037**
3	8327	394	29.5	0.35	**1.0035**	23.0	0.28	**1.0028**	28.1	0.34	**1.0034**
4	10440	523	31.4	0.30	**1.0030**	25.0	0.24	**1.0024**	31.6	0.30	**1.0030**
5	8660	606	29.8	0.34	**1.0034**	23.3	0.27	**1.0027**	28.6	0.33	**1.0033**
6	10535	581	31.5	0.30	**1.0030**	25.1	0.24	**1.0024**	31.8	0.30	**1.0030**
7	10259	768	31.3	0.30	**1.0030**	24.8	0.24	**1.0024**	31.3	0.31	**1.0031**
8	10440	1490	31.4	0.30	**1.0030**	25.0	0.24	**1.0024**	31.6	0.30	**1.0030**
9	9861	1490	30.9	0.31	**1.0031**	24.5	0.25	**1.0025**	30.6	0.31	**1.0031**
10	10226	1490	31.2	0.31	**1.0031**	24.8	0.24	**1.0024**	31.3	0.31	**1.0031**
11	11136	1490	32.1	0.29	**1.0029**	25.7	0.23	**1.0023**	32.8	0.29	**1.0029**
12	12490	1490	33.3	0.27	**1.0027**	27.0	0.22	**1.0022**	35.0	0.28	**1.0028**
13	10706	1490	31.7	0.30	**1.0030**	25.3	0.24	**1.0024**	32.1	0.30	**1.0030**
14	11907	1490	32.8	0.28	**1.0028**	26.5	0.22	**1.0022**	34.1	0.29	**1.0029**
15	12845	1852	33.6	0.26	**1.0026**	27.4	0.21	**1.0021**	35.6	0.28	**1.0028**
16	14318	1852	35.0	0.24	**1.0024**	28.8	0.20	**1.0020**	38.1	0.27	**1.0027**

	CALCULATION SHEET	PROJECT NO.: M.ENG
CLIENT: MCGILL	PREPARED: A.G.	PAGE OF
PROJECT: CBFs Building 7H	CODE: NA	
SUBJECT: IDA Results and Analysis	DATE: 2012-12-01	1 13

Building	7H		A	768	mm^2	*Area of brace member*
Bay Width	7500	mm	P_y	300	kN	*Yield force*
Height	7000	mm	δ_y	20.0	mm	*Yield deformation*
Brace Length	10259	mm	Δ_y	27.3	mm	*Yield drift*
T	0.779	sec				

Ground motion		Scaling factor		Δbrace		% Drift
GM:	1	SF:	0.2	brace:	1.681	0.024
GM:	1	SF:	0.4	brace:	3.517	0.0502
GM:	1	SF:	0.6	brace:	5.285	0.0755
GM:	1	SF:	0.8	brace:	6.937	0.0991
GM:	1	SF:	1	brace:	8.499	0.1214
GM:	1	SF:	1.2	brace:	9.996	0.1428
GM:	1	SF:	1.4	brace:	11.45	0.1635
GM:	1	SF:	1.6	brace:	12.85	0.1836
GM:	1	SF:	1.8	brace:	14.23	0.2033
GM:	1	SF:	2	brace:	15.58	0.2226
GM:	1	SF:	2.2	brace:	16.91	0.2415
GM:	1	SF:	2.4	brace:	18.2	0.2601
GM:	1	SF:	2.6	brace:	19.46	0.278
GM:	1	SF:	2.8	brace:	20.63	0.2947
GM:	1	SF:	3	brace:	21.65	0.3093
GM:	1	SF:	3.2	brace:	22.45	0.3208
GM:	1	SF:	3.4	brace:	22.98	0.3283
GM:	1	SF:	3.6	brace:	24.56	0.3509
GM:	1	SF:	3.8	brace:	26.29	0.3755
GM:	1	SF:	4	brace:	28.1	0.4014
GM:	1	SF:	4.2	brace:	29.96	0.428
GM:	1	SF:	4.4	brace:	31.86	0.4551
GM:	1	SF:	4.6	brace:	33.77	0.4824
GM:	1	SF:	4.8	brace:	35.68	0.5097
GM:	1	SF:	5	brace:	37.56	0.5366
GM:	1	SF:	5.2	brace:	39.42	0.5631
GM:	1	SF:	5.4	brace:	41.21	0.5887
GM:	1	SF:	5.6	brace:	42.92	0.6132
GM:	1	SF:	5.8	brace:	44.53	0.6362
GM:	1	SF:	6	brace:	46.03	0.6576

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Ground motion		Scaling factor			Δbrace	% Drift
GM:	2	SF:	0.2	brace:	3.14	0.0449
GM:	2	SF:	0.4	brace:	7.171	0.1024
GM:	2	SF:	0.6	brace:	11.44	0.1634
GM:	2	SF:	0.8	brace:	15.97	0.2282
GM:	2	SF:	1	brace:	20.06	0.2866
GM:	2	SF:	1.2	brace:	24.24	0.3463
GM:	2	SF:	1.4	brace:	27.73	0.3962
GM:	2	SF:	1.6	brace:	33.62	0.4803
GM:	2	SF:	1.8	brace:	39.97	0.571
GM:	2	SF:	2	brace:	45.06	0.6438
GM:	2	SF:	2.2	brace:	48.61	0.6944
GM:	2	SF:	2.4	brace:	51.16	0.7308
GM:	2	SF:	2.6	brace:	53.09	0.7584
GM:	2	SF:	2.8	brace:	54.57	0.7795
GM:	2	SF:	3	brace:	54.99	0.7856
GM:	2	SF:	3.2	brace:	56.89	0.8127
GM:	2	SF:	3.4	brace:	59.58	0.8512
GM:	2	SF:	3.6	brace:	64.96	0.928
GM:	2	SF:	3.8	brace:	70.44	1.0062
GM:	2	SF:	4	brace:	76	1.0857
GM:	2	SF:	4.2	brace:	81.65	1.1665
GM:	2	SF:	4.4	brace:	87.38	1.2483
GM:	2	SF:	4.6	brace:	93.18	1.3311
GM:	2	SF:	4.8	brace:	99.01	1.4144
GM:	2	SF:	5	brace:	104.9	1.498
GM:	2	SF:	5.2	brace:	111.2	1.5887
GM:	2	SF:	5.4	brace:	118	1.685
GM:	2	SF:	5.6	brace:	124.7	1.7812
GM:	2	SF:	5.8	brace:	131.3	1.8764
GM:	2	SF:	6	brace:	138.2	1.9743
GM:	3	SF:	0.2	brace:	1.983	0.0283
GM:	3	SF:	0.4	brace:	4.094	0.0585
GM:	3	SF:	0.6	brace:	6.835	0.0976
GM:	3	SF:	0.8	brace:	9.738	0.1391
GM:	3	SF:	1	brace:	12.52	0.1788
GM:	3	SF:	1.2	brace:	15.18	0.2168
GM:	3	SF:	1.4	brace:	17.73	0.2532
GM:	3	SF:	1.6	brace:	20.15	0.2879
GM:	3	SF:	1.8	brace:	22.28	0.3183
GM:	3	SF:	2	brace:	23.8	0.3401
GM:	3	SF:	2.2	brace:	24.63	0.3519
GM:	3	SF:	2.4	brace:	26.47	0.3782
GM:	3	SF:	2.6	brace:	27.93	0.399
GM:	3	SF:	2.8	brace:	30.06	0.4294
GM:	3	SF:	3	brace:	32.49	0.4641
GM:	3	SF:	3.2	brace:	34.42	0.4917
GM:	3	SF:	3.4	brace:	35.65	0.5093
GM:	3	SF:	3.6	brace:	36.02	0.5145
GM:	3	SF:	3.8	brace:	35.58	0.5083
GM:	3	SF:	4	brace:	34.57	0.4938
GM:	3	SF:	4.2	brace:	34.18	0.4883
GM:	3	SF:	4.4	brace:	37.05	0.5293
GM:	3	SF:	4.6	brace:	40.02	0.5717
GM:	3	SF:	4.8	brace:	43.08	0.6154
GM:	3	SF:	5	brace:	46.24	0.6605
GM:	3	SF:	5.2	brace:	49.47	0.7067
GM:	3	SF:	5.4	brace:	52.78	0.7541
GM:	3	SF:	5.6	brace:	56.18	0.8026
GM:	3	SF:	5.8	brace:	59.65	0.8521
GM:	3	SF:	6	brace:	63.2	0.9029

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Ground motion		Scaling factor		Δ_{brace}		% Drift
GM:	4	SF:	0.2	brace:	1.256	0.0179
GM:	4	SF:	0.4	brace:	2.537	0.0362
GM:	4	SF:	0.6	brace:	4.437	0.0634
GM:	4	SF:	0.8	brace:	6.606	0.0944
GM:	4	SF:	1	brace:	8.798	0.1257
GM:	4	SF:	1.2	brace:	10.87	0.1553
GM:	4	SF:	1.4	brace:	12.8	0.1829
GM:	4	SF:	1.6	brace:	14.64	0.2092
GM:	4	SF:	1.8	brace:	16.51	0.2359
GM:	4	SF:	2	brace:	18.22	0.2603
GM:	4	SF:	2.2	brace:	20.11	0.2873
GM:	4	SF:	2.4	brace:	21.96	0.3138
GM:	4	SF:	2.6	brace:	23.56	0.3366
GM:	4	SF:	2.8	brace:	24.73	0.3533
GM:	4	SF:	3	brace:	26.9	0.3843
GM:	4	SF:	3.2	brace:	29.22	0.4175
GM:	4	SF:	3.4	brace:	31.18	0.4454
GM:	4	SF:	3.6	brace:	32.46	0.4637
GM:	4	SF:	3.8	brace:	32.51	0.4644
GM:	4	SF:	4	brace:	31.07	0.4438
GM:	4	SF:	4.2	brace:	30.26	0.4323
GM:	4	SF:	4.4	brace:	32.37	0.4624
GM:	4	SF:	4.6	brace:	34.56	0.4937
GM:	4	SF:	4.8	brace:	36.83	0.5261
GM:	4	SF:	5	brace:	42.49	0.607
GM:	4	SF:	5.2	brace:	42.11	0.6015
GM:	4	SF:	5.4	brace:	43.91	0.6273
GM:	4	SF:	5.6	brace:	46.26	0.6608
GM:	4	SF:	5.8	brace:	48.68	0.6954
GM:	4	SF:	6	brace:	53.78	0.7683
GM:	5	SF:	0.2	brace:	3.247	0.0464
GM:	5	SF:	0.4	brace:	7.073	0.101
GM:	5	SF:	0.6	brace:	11.09	0.1585
GM:	5	SF:	0.8	brace:	15.39	0.2199
GM:	5	SF:	1	brace:	19.6	0.28
GM:	5	SF:	1.2	brace:	24.08	0.344
GM:	5	SF:	1.4	brace:	28.92	0.4132
GM:	5	SF:	1.6	brace:	35.22	0.5031
GM:	5	SF:	1.8	brace:	39.05	0.5579
GM:	5	SF:	2	brace:	42.64	0.6091
GM:	5	SF:	2.2	brace:	46.88	0.6697
GM:	5	SF:	2.4	brace:	49.89	0.7127
GM:	5	SF:	2.6	brace:	56.46	0.8065
GM:	5	SF:	2.8	brace:	63.07	0.901
GM:	5	SF:	3	brace:	69.77	0.9968
GM:	5	SF:	3.2	brace:	76.45	1.0921
GM:	5	SF:	3.4	brace:	82.33	1.1762
GM:	5	SF:	3.6	brace:	87.07	1.2438
GM:	5	SF:	3.8	brace:	91.33	1.3047
GM:	5	SF:	4	brace:	95.29	1.3613
GM:	5	SF:	4.2	brace:	98.99	1.4141
GM:	5	SF:	4.4	brace:	102.4	1.4631
GM:	5	SF:	4.6	brace:	105.4	1.5062
GM:	5	SF:	4.8	brace:	108	1.5436
GM:	5	SF:	5	brace:	110.1	1.5733
GM:	5	SF:	5.2	brace:	110.5	1.579
GM:	5	SF:	5.4	brace:	112	1.5996
GM:	5	SF:	5.6	brace:	113.2	1.6167
GM:	5	SF:	5.8	brace:	114.1	1.6303
GM:	5	SF:	6	brace:	114.9	1.6409

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Ground motion		Scaling factor		Δbrace		% Drift
GM:	6	SF:	0.2	brace:	2.64	0.038
GM:	6	SF:	0.4	brace:	5.03	0.072
GM:	6	SF:	0.6	brace:	7.92	0.113
GM:	6	SF:	0.8	brace:	12.4	0.177
GM:	6	SF:	1	brace:	16.3	0.232
GM:	6	SF:	1.2	brace:	19.3	0.276
GM:	6	SF:	1.4	brace:	21.7	0.31
GM:	6	SF:	1.6	brace:	27.5	0.393
GM:	6	SF:	1.8	brace:	33.3	0.475
GM:	6	SF:	2	brace:	33.5	0.478
GM:	6	SF:	2.2	brace:	40.5	0.578
GM:	6	SF:	2.4	brace:	47.8	0.683
GM:	6	SF:	2.6	brace:	59	0.843
GM:	6	SF:	2.8	brace:	63.8	0.912
GM:	6	SF:	3	brace:	70.6	1.009
GM:	6	SF:	3.2	brace:	61.3	0.876
GM:	6	SF:	3.4	brace:	67.6	0.966
GM:	6	SF:	3.6	brace:	82.1	1.173
GM:	6	SF:	3.8	brace:	86.8	1.24
GM:	6	SF:	4	brace:	90.1	1.288
GM:	6	SF:	4.2	brace:	120	1.713
GM:	6	SF:	4.4	brace:	93.5	1.336
GM:	6	SF:	4.6	brace:	103	1.472
GM:	6	SF:	4.8	brace:	110	1.569
GM:	6	SF:	5	brace:	117	1.669
GM:	6	SF:	5.2	brace:	126	1.801
GM:	6	SF:	5.4	brace:	137	1.963
GM:	6	SF:	5.6	brace:	152	2.168
GM:	6	SF:	5.8	brace:	168	2.396
GM:	6	SF:	6	brace:	182	2.605
GM:	7	SF:	0.2	brace:	2.44	0.035
GM:	7	SF:	0.4	brace:	4.68	0.067
GM:	7	SF:	0.6	brace:	6.26	0.089
GM:	7	SF:	0.8	brace:	7.4	0.106
GM:	7	SF:	1	brace:	9.09	0.13
GM:	7	SF:	1.2	brace:	11	0.157
GM:	7	SF:	1.4	brace:	13.5	0.193
GM:	7	SF:	1.6	brace:	16	0.228
GM:	7	SF:	1.8	brace:	18.4	0.263
GM:	7	SF:	2	brace:	21.1	0.301
GM:	7	SF:	2.2	brace:	26.4	0.377
GM:	7	SF:	2.4	brace:	38.1	0.544
GM:	7	SF:	2.6	brace:	44	0.629
GM:	7	SF:	2.8	brace:	46	0.657
GM:	7	SF:	3	brace:	61.9	0.884
GM:	7	SF:	3.2	brace:	79.8	1.14
GM:	7	SF:	3.4	brace:	93.3	1.333
GM:	7	SF:	3.6	brace:	102	1.458
GM:	7	SF:	3.8	brace:	102	1.457
GM:	7	SF:	4	brace:	99.6	1.423
GM:	7	SF:	4.2	brace:	101	1.449
GM:	7	SF:	4.4	brace:	101	1.446
GM:	7	SF:	4.6	brace:	104	1.484
GM:	7	SF:	4.8	brace:	105	1.498
GM:	7	SF:	5	brace:	108	1.539
GM:	7	SF:	5.2	brace:	108	1.543
GM:	7	SF:	5.4	brace:	123	1.76
GM:	7	SF:	5.6	brace:	145	2.077
GM:	7	SF:	5.8	brace:	160	2.288
GM:	7	SF:	6	brace:	160	2.28

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Ground motion		Scaling factor		Δbrace		% Drift
GM:	8	SF:	0.2	brace:	3.846	0.0549
GM:	8	SF:	0.4	brace:	7.552	0.1079
GM:	8	SF:	0.6	brace:	10.66	0.1522
GM:	8	SF:	0.8	brace:	13.38	0.1912
GM:	8	SF:	1	brace:	16.65	0.2378
GM:	8	SF:	1.2	brace:	20.29	0.2898
GM:	8	SF:	1.4	brace:	25.7	0.3671
GM:	8	SF:	1.6	brace:	33.47	0.4781
GM:	8	SF:	1.8	brace:	41.17	0.5882
GM:	8	SF:	2	brace:	44.83	0.6404
GM:	8	SF:	2.2	brace:	50.81	0.7258
GM:	8	SF:	2.4	brace:	52.97	0.7567
GM:	8	SF:	2.6	brace:	51.64	0.7377
GM:	8	SF:	2.8	brace:	49.38	0.7054
GM:	8	SF:	3	brace:	60.57	0.8652
GM:	8	SF:	3.2	brace:	70.45	1.0064
GM:	8	SF:	3.4	brace:	74.21	1.0601
GM:	8	SF:	3.6	brace:	78.65	1.1235
GM:	8	SF:	3.8	brace:	83.47	1.1925
GM:	8	SF:	4	brace:	85.78	1.2254
GM:	8	SF:	4.2	brace:	90.03	1.2862
GM:	8	SF:	4.4	brace:	99.23	1.4176
GM:	8	SF:	4.6	brace:	107.7	1.5391
GM:	8	SF:	4.8	brace:	117.3	1.6761
GM:	8	SF:	5	brace:	126.1	1.8008
GM:	8	SF:	5.2	brace:	129.4	1.8493
GM:	8	SF:	5.4	brace:	133.6	1.9087
GM:	8	SF:	5.6	brace:	122.4	1.749
GM:	8	SF:	5.8	brace:	124.4	1.7764
GM:	8	SF:	6	brace:	124.1	1.773
GM:	9	SF:	0.2	brace:	2.769	0.0396
GM:	9	SF:	0.4	brace:	6.954	0.0993
GM:	9	SF:	0.6	brace:	12.21	0.1744
GM:	9	SF:	0.8	brace:	17.82	0.2545
GM:	9	SF:	1	brace:	23.26	0.3322
GM:	9	SF:	1.2	brace:	34.15	0.4878
GM:	9	SF:	1.4	brace:	34.87	0.4981
GM:	9	SF:	1.6	brace:	45.06	0.6437
GM:	9	SF:	1.8	brace:	55.57	0.7939
GM:	9	SF:	2	brace:	55.75	0.7965
GM:	9	SF:	2.2	brace:	60.49	0.8641
GM:	9	SF:	2.4	brace:	67.15	0.9592
GM:	9	SF:	2.6	brace:	72.73	1.039
GM:	9	SF:	2.8	brace:	77.81	1.1115
GM:	9	SF:	3	brace:	81.43	1.1633
GM:	9	SF:	3.2	brace:	87.82	1.2546
GM:	9	SF:	3.4	brace:	95.73	1.3675
GM:	9	SF:	3.6	brace:	103.8	1.4833
GM:	9	SF:	3.8	brace:	111.6	1.5945
GM:	9	SF:	4	brace:	119	1.7002
GM:	9	SF:	4.2	brace:	126.1	1.8012
GM:	9	SF:	4.4	brace:	132.8	1.8969
GM:	9	SF:	4.6	brace:	139.3	1.9894
GM:	9	SF:	4.8	brace:	143.4	2.0491
GM:	9	SF:	5	brace:	149.8	2.1395
GM:	9	SF:	5.2	brace:	155.7	2.2245
GM:	9	SF:	5.4	brace:	161.4	2.3054
GM:	9	SF:	5.6	brace:	166.8	2.3824
GM:	9	SF:	5.8	brace:	172	2.4566
GM:	9	SF:	6	brace:	177	2.5285

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Ground motion		Scaling factor		Δbrace		% Drift
GM:	10	SF:	0.2	brace:	2.766	0.0395
GM:	10	SF:	0.4	brace:	5.95	0.085
GM:	10	SF:	0.6	brace:	8.801	0.1257
GM:	10	SF:	0.8	brace:	11.77	0.1681
GM:	10	SF:	1	brace:	15.47	0.221
GM:	10	SF:	1.2	brace:	19.44	0.2777
GM:	10	SF:	1.4	brace:	24.16	0.3452
GM:	10	SF:	1.6	brace:	30.24	0.4319
GM:	10	SF:	1.8	brace:	36.6	0.5229
GM:	10	SF:	2	brace:	41.82	0.5975
GM:	10	SF:	2.2	brace:	39.94	0.5706
GM:	10	SF:	2.4	brace:	45.05	0.6435
GM:	10	SF:	2.6	brace:	51.8	0.74
GM:	10	SF:	2.8	brace:	54.67	0.781
GM:	10	SF:	3	brace:	55.73	0.7961
GM:	10	SF:	3.2	brace:	58.3	0.8328
GM:	10	SF:	3.4	brace:	65.75	0.9392
GM:	10	SF:	3.6	brace:	71.09	1.0156
GM:	10	SF:	3.8	brace:	71.86	1.0265
GM:	10	SF:	4	brace:	64.15	0.9165
GM:	10	SF:	4.2	brace:	53.36	0.7622
GM:	10	SF:	4.4	brace:	48.39	0.6914
GM:	10	SF:	4.6	brace:	48.58	0.694
GM:	10	SF:	4.8	brace:	63.1	0.9015
GM:	10	SF:	5	brace:	66.98	0.9568
GM:	10	SF:	5.2	brace:	74.05	1.0578
GM:	10	SF:	5.4	brace:	83.49	1.1928
GM:	10	SF:	5.6	brace:	95.39	1.3627
GM:	10	SF:	5.8	brace:	100.5	1.4354
GM:	10	SF:	6	brace:	107.8	1.5401
GM:	11	SF:	0.2	brace:	0.834	0.0119
GM:	11	SF:	0.4	brace:	1.622	0.0232
GM:	11	SF:	0.6	brace:	2.277	0.0325
GM:	11	SF:	0.8	brace:	3.006	0.0429
GM:	11	SF:	1	brace:	3.776	0.0539
GM:	11	SF:	1.2	brace:	4.601	0.0657
GM:	11	SF:	1.4	brace:	5.458	0.078
GM:	11	SF:	1.6	brace:	6.283	0.0898
GM:	11	SF:	1.8	brace:	7.075	0.1011
GM:	11	SF:	2	brace:	7.753	0.1108
GM:	11	SF:	2.2	brace:	8.769	0.1253
GM:	11	SF:	2.4	brace:	9.67	0.1381
GM:	11	SF:	2.6	brace:	10.61	0.1516
GM:	11	SF:	2.8	brace:	11.67	0.1667
GM:	11	SF:	3	brace:	12.69	0.1812
GM:	11	SF:	3.2	brace:	13.53	0.1933
GM:	11	SF:	3.4	brace:	14.22	0.2032
GM:	11	SF:	3.6	brace:	14.79	0.2113
GM:	11	SF:	3.8	brace:	15.24	0.2178
GM:	11	SF:	4	brace:	16.17	0.231
GM:	11	SF:	4.2	brace:	17.26	0.2465
GM:	11	SF:	4.4	brace:	18.56	0.2652
GM:	11	SF:	4.6	brace:	19.8	0.2828
GM:	11	SF:	4.8	brace:	21.06	0.3008
GM:	11	SF:	5	brace:	22.29	0.3184
GM:	11	SF:	5.2	brace:	23.47	0.3353
GM:	11	SF:	5.4	brace:	24.58	0.3511
GM:	11	SF:	5.6	brace:	25.56	0.3651
GM:	11	SF:	5.8	brace:	26.38	0.3768
GM:	11	SF:	6	brace:	26.83	0.3832

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Ground motion		Scaling factor		Δbrace		% Drift
GM:	12	SF:	0.2	brace:	3.402	0.0486
GM:	12	SF:	0.4	brace:	9.648	0.1378
GM:	12	SF:	0.6	brace:	15.49	0.2213
GM:	12	SF:	0.8	brace:	21.76	0.3108
GM:	12	SF:	1	brace:	27.97	0.3996
GM:	12	SF:	1.2	brace:	31.35	0.4478
GM:	12	SF:	1.4	brace:	26.02	0.3718
GM:	12	SF:	1.6	brace:	30.75	0.4392
GM:	12	SF:	1.8	brace:	34.42	0.4917
GM:	12	SF:	2	brace:	43.11	0.6158
GM:	12	SF:	2.2	brace:	54.33	0.7761
GM:	12	SF:	2.4	brace:	52.54	0.7505
GM:	12	SF:	2.6	brace:	52.18	0.7454
GM:	12	SF:	2.8	brace:	60.8	0.8686
GM:	12	SF:	3	brace:	66.16	0.9451
GM:	12	SF:	3.2	brace:	69.61	0.9945
GM:	12	SF:	3.4	brace:	74.1	1.0585
GM:	12	SF:	3.6	brace:	79.67	1.1382
GM:	12	SF:	3.8	brace:	87.25	1.2464
GM:	12	SF:	4	brace:	97.89	1.3984
GM:	12	SF:	4.2	brace:	105.3	1.5045
GM:	12	SF:	4.4	brace:	111.2	1.5879
GM:	12	SF:	4.6	brace:	115	1.6428
GM:	12	SF:	4.8	brace:	118.4	1.691
GM:	12	SF:	5	brace:	121.2	1.7314
GM:	12	SF:	5.2	brace:	138.9	1.9841
GM:	12	SF:	5.4	brace:	131.3	1.8763
GM:	12	SF:	5.6	brace:	144.7	2.0669
GM:	12	SF:	5.8	brace:	167.5	2.3925
GM:	12	SF:	6	brace:	179.2	2.5607
GM:	13	SF:	0.2	brace:	4.359	0.0623
GM:	13	SF:	0.4	brace:	8.022	0.1146
GM:	13	SF:	0.6	brace:	11.78	0.1683
GM:	13	SF:	0.8	brace:	15.08	0.2155
GM:	13	SF:	1	brace:	17.96	0.2566
GM:	13	SF:	1.2	brace:	19.75	0.2821
GM:	13	SF:	1.4	brace:	20.56	0.2938
GM:	13	SF:	1.6	brace:	25.73	0.3676
GM:	13	SF:	1.8	brace:	33.83	0.4833
GM:	13	SF:	2	brace:	35.04	0.5005
GM:	13	SF:	2.2	brace:	50.35	0.7192
GM:	13	SF:	2.4	brace:	48.34	0.6906
GM:	13	SF:	2.6	brace:	42.59	0.6084
GM:	13	SF:	2.8	brace:	49.46	0.7066
GM:	13	SF:	3	brace:	57.1	0.8157
GM:	13	SF:	3.2	brace:	63.93	0.9133
GM:	13	SF:	3.4	brace:	68.32	0.9759
GM:	13	SF:	3.6	brace:	72.35	1.0335
GM:	13	SF:	3.8	brace:	70.95	1.0135
GM:	13	SF:	4	brace:	82.46	1.178
GM:	13	SF:	4.2	brace:	89.35	1.2764
GM:	13	SF:	4.4	brace:	93.3	1.3328
GM:	13	SF:	4.6	brace:	98.9	1.4128
GM:	13	SF:	4.8	brace:	109.4	1.5634
GM:	13	SF:	5	brace:	128.1	1.8303
GM:	13	SF:	5.2	brace:	133.4	1.9051
GM:	13	SF:	5.4	brace:	149.2	2.1308
GM:	13	SF:	5.6	brace:	167	2.3857
GM:	13	SF:	5.8	brace:	178.6	2.5515
GM:	13	SF:	6	brace:	188	2.6854

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Ground motion		Scaling factor		Δbrace		% Drift
GM:	14	SF:	0.2	brace:	0.025	0.0004
GM:	14	SF:	0.4	brace:	0.025	0.0004
GM:	14	SF:	0.6	brace:	0.025	0.0004
GM:	14	SF:	0.8	brace:	0.025	0.0004
GM:	14	SF:	1	brace:	0.025	0.0004
GM:	14	SF:	1.2	brace:	0.025	0.0004
GM:	14	SF:	1.4	brace:	0.025	0.0004
GM:	14	SF:	1.6	brace:	0.025	0.0004
GM:	14	SF:	1.8	brace:	0.025	0.0004
GM:	14	SF:	2	brace:	0.025	0.0004
GM:	14	SF:	2.2	brace:	0.025	0.0004
GM:	14	SF:	2.4	brace:	0.025	0.0004
GM:	14	SF:	2.6	brace:	0.025	0.0004
GM:	14	SF:	2.8	brace:	0.025	0.0004
GM:	14	SF:	3	brace:	0.025	0.0004
GM:	14	SF:	3.2	brace:	0.025	0.0004
GM:	14	SF:	3.4	brace:	0.025	0.0004
GM:	14	SF:	3.6	brace:	0.025	0.0004
GM:	14	SF:	3.8	brace:	0.025	0.0004
GM:	14	SF:	4	brace:	0.025	0.0004
GM:	14	SF:	4.2	brace:	0.025	0.0004
GM:	14	SF:	4.4	brace:	0.025	0.0004
GM:	14	SF:	4.6	brace:	0.025	0.0004
GM:	14	SF:	4.8	brace:	0.025	0.0004
GM:	14	SF:	5	brace:	0.025	0.0004
GM:	14	SF:	5.2	brace:	0.025	0.0004
GM:	14	SF:	5.4	brace:	0.025	0.0004
GM:	14	SF:	5.6	brace:	0.025	0.0004
GM:	14	SF:	5.8	brace:	0.025	0.0004
GM:	14	SF:	6	brace:	0.025	0.0004
GM:	15	SF:	0.2	brace:	2.34	0.0334
GM:	15	SF:	0.4	brace:	4.015	0.0574
GM:	15	SF:	0.6	brace:	5.934	0.0848
GM:	15	SF:	0.8	brace:	7.667	0.1095
GM:	15	SF:	1	brace:	9.325	0.1332
GM:	15	SF:	1.2	brace:	11	0.1572
GM:	15	SF:	1.4	brace:	12.75	0.1822
GM:	15	SF:	1.6	brace:	14.61	0.2088
GM:	15	SF:	1.8	brace:	16.59	0.2371
GM:	15	SF:	2	brace:	18.69	0.267
GM:	15	SF:	2.2	brace:	20.9	0.2986
GM:	15	SF:	2.4	brace:	23.25	0.3321
GM:	15	SF:	2.6	brace:	27.52	0.3932
GM:	15	SF:	2.8	brace:	33.01	0.4716
GM:	15	SF:	3	brace:	38.03	0.5433
GM:	15	SF:	3.2	brace:	42.17	0.6025
GM:	15	SF:	3.4	brace:	43.03	0.6148
GM:	15	SF:	3.6	brace:	41.32	0.5902
GM:	15	SF:	3.8	brace:	51.29	0.7327
GM:	15	SF:	4	brace:	66.06	0.9437
GM:	15	SF:	4.2	brace:	74.33	1.0618
GM:	15	SF:	4.4	brace:	79.11	1.1302
GM:	15	SF:	4.6	brace:	85.77	1.2252
GM:	15	SF:	4.8	brace:	92.46	1.3208
GM:	15	SF:	5	brace:	94.64	1.352
GM:	15	SF:	5.2	brace:	98.43	1.4062
GM:	15	SF:	5.4	brace:	101.7	1.4525
GM:	15	SF:	5.6	brace:	107.8	1.5396
GM:	15	SF:	5.8	brace:	107.3	1.5325
GM:	15	SF:	6	brace:	95.08	1.3582

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				M.ENG	
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Ground motion		Scaling factor			Δbrace	% Drift
GM:	16	SF:	0.2	brace:	2.474	0.0353
GM:	16	SF:	0.4	brace:	5.684	0.0812
GM:	16	SF:	0.6	brace:	9.832	0.1405
GM:	16	SF:	0.8	brace:	15.85	0.2264
GM:	16	SF:	1	brace:	21.97	0.3139
GM:	16	SF:	1.2	brace:	25.58	0.3654
GM:	16	SF:	1.4	brace:	31.01	0.4431
GM:	16	SF:	1.6	brace:	33.03	0.4718
GM:	16	SF:	1.8	brace:	35.86	0.5123
GM:	16	SF:	2	brace:	38.91	0.5559
GM:	16	SF:	2.2	brace:	41.11	0.5872
GM:	16	SF:	2.4	brace:	42.59	0.6085
GM:	16	SF:	2.6	brace:	43.4	0.6199
GM:	16	SF:	2.8	brace:	43.72	0.6245
GM:	16	SF:	3	brace:	44.03	0.629
GM:	16	SF:	3.2	brace:	45.21	0.6458
GM:	16	SF:	3.4	brace:	47.82	0.6832
GM:	16	SF:	3.6	brace:	51.37	0.7339
GM:	16	SF:	3.8	brace:	56.91	0.813
GM:	16	SF:	4	brace:	63.5	0.9072
GM:	16	SF:	4.2	brace:	63.2	0.9028
GM:	16	SF:	4.4	brace:	60.01	0.8573
GM:	16	SF:	4.6	brace:	59.92	0.856
GM:	16	SF:	4.8	brace:	60.15	0.8593
GM:	16	SF:	5	brace:	61.35	0.8764
GM:	16	SF:	5.2	brace:	72.49	1.0356
GM:	16	SF:	5.4	brace:	76.66	1.0952
GM:	16	SF:	5.6	brace:	72.42	1.0345
GM:	16	SF:	5.8	brace:	82.99	1.1855
GM:	16	SF:	6	brace:	79.43	1.1347
GM:	17	SF:	0.2	brace:	0.983	0.014
GM:	17	SF:	0.4	brace:	1.701	0.0243
GM:	17	SF:	0.6	brace:	1.88	0.0269
GM:	17	SF:	0.8	brace:	2.583	0.0369
GM:	17	SF:	1	brace:	2.385	0.0341
GM:	17	SF:	1.2	brace:	3.905	0.0558
GM:	17	SF:	1.4	brace:	3.419	0.0488
GM:	17	SF:	1.6	brace:	3.936	0.0562
GM:	17	SF:	1.8	brace:	5.006	0.0715
GM:	17	SF:	2	brace:	5.136	0.0734
GM:	17	SF:	2.2	brace:	51.91	0.7416
GM:	17	SF:	2.4	brace:	49.8	0.7114
GM:	17	SF:	2.6	brace:	53.57	0.7653
GM:	17	SF:	2.8	brace:	48.04	0.6863
GM:	17	SF:	3	brace:	59.59	0.8512
GM:	17	SF:	3.2	brace:	62.77	0.8967
GM:	17	SF:	3.4	brace:	66.45	0.9493
GM:	17	SF:	3.6	brace:	72.57	1.0367
GM:	17	SF:	3.8	brace:	141.1	2.0158
GM:	17	SF:	4	brace:	139.8	1.9976
GM:	17	SF:	4.2	brace:	136.7	1.9531
GM:	17	SF:	4.4	brace:	162.7	2.3243
GM:	17	SF:	4.6	brace:	140.4	2.0056
GM:	17	SF:	4.8	brace:	139.4	1.992
GM:	17	SF:	5	brace:	149.6	2.1375
GM:	17	SF:	5.2	brace:	168.3	2.4049
GM:	17	SF:	5.4	brace:	177.8	2.5402
GM:	17	SF:	5.6	brace:	192.7	2.7522
GM:	17	SF:	5.8	brace:	165.1	2.359
GM:	17	SF:	6	brace:	172.8	2.4681

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Ground motion		Scaling factor		Δbrace		% Drift
GM:	18	SF:	0.2	brace:	1.282	0.0183
GM:	18	SF:	0.4	brace:	2.513	0.0359
GM:	18	SF:	0.6	brace:	3.488	0.0498
GM:	18	SF:	0.8	brace:	4.002	0.0572
GM:	18	SF:	1	brace:	4.844	0.0692
GM:	18	SF:	1.2	brace:	5.786	0.0827
GM:	18	SF:	1.4	brace:	6.722	0.096
GM:	18	SF:	1.6	brace:	7.569	0.1081
GM:	18	SF:	1.8	brace:	5.101	0.0729
GM:	18	SF:	2	brace:	9.038	0.1291
GM:	18	SF:	2.2	brace:	27.5	0.3928
GM:	18	SF:	2.4	brace:	35.02	0.5002
GM:	18	SF:	2.6	brace:	39.9	0.57
GM:	18	SF:	2.8	brace:	46.33	0.6619
GM:	18	SF:	3	brace:	58.07	0.8296
GM:	18	SF:	3.2	brace:	48.18	0.6883
GM:	18	SF:	3.4	brace:	75.33	1.0762
GM:	18	SF:	3.6	brace:	76.77	1.0968
GM:	18	SF:	3.8	brace:	74.7	1.0671
GM:	18	SF:	4	brace:	86.34	1.2334
GM:	18	SF:	4.2	brace:	90.29	1.2899
GM:	18	SF:	4.4	brace:	104.4	1.4907
GM:	18	SF:	4.6	brace:	116.4	1.6633
GM:	18	SF:	4.8	brace:	106.9	1.5273
GM:	18	SF:	5	brace:	113.2	1.6177
GM:	18	SF:	5.2	brace:	116.7	1.6675
GM:	18	SF:	5.4	brace:	115.9	1.6562
GM:	18	SF:	5.6	brace:	121.2	1.7321
GM:	18	SF:	5.8	brace:	129.4	1.8479
GM:	18	SF:	6	brace:	134.8	1.9251
GM:	19	SF:	0.2	brace:	3.874	0.0553
GM:	19	SF:	0.4	brace:	6.834	0.0976
GM:	19	SF:	0.6	brace:	9.1	0.13
GM:	19	SF:	0.8	brace:	10.51	0.1502
GM:	19	SF:	1	brace:	12.38	0.1769
GM:	19	SF:	1.2	brace:	13.78	0.1969
GM:	19	SF:	1.4	brace:	14.91	0.213
GM:	19	SF:	1.6	brace:	16.88	0.2412
GM:	19	SF:	1.8	brace:	18.76	0.268
GM:	19	SF:	2	brace:	20.61	0.2945
GM:	19	SF:	2.2	brace:	22.53	0.3219
GM:	19	SF:	2.4	brace:	24.55	0.3508
GM:	19	SF:	2.6	brace:	26.71	0.3815
GM:	19	SF:	2.8	brace:	28.98	0.4141
GM:	19	SF:	3	brace:	31.35	0.4478
GM:	19	SF:	3.2	brace:	33.77	0.4824
GM:	19	SF:	3.4	brace:	40.2	0.5743
GM:	19	SF:	3.6	brace:	48.31	0.6901
GM:	19	SF:	3.8	brace:	59.37	0.8481
GM:	19	SF:	4	brace:	69.65	0.995
GM:	19	SF:	4.2	brace:	71.31	1.0187
GM:	19	SF:	4.4	brace:	71.77	1.0253
GM:	19	SF:	4.6	brace:	77.03	1.1004
GM:	19	SF:	4.8	brace:	83.93	1.1991
GM:	19	SF:	5	brace:	89.86	1.2836
GM:	19	SF:	5.2	brace:	92.59	1.3228
GM:	19	SF:	5.4	brace:	91.97	1.3138
GM:	19	SF:	5.6	brace:	89.15	1.2736
GM:	19	SF:	5.8	brace:	85.46	1.2209
GM:	19	SF:	6	brace:	81.62	1.1661

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Ground motion		Scaling factor		Δbrace		% Drift
GM:	20	SF:	0.2	brace:	0.882	0.0126
GM:	20	SF:	0.4	brace:	1.353	0.0193
GM:	20	SF:	0.6	brace:	2.69	0.0384
GM:	20	SF:	0.8	brace:	3.657	0.0522
GM:	20	SF:	1	brace:	3.195	0.0456
GM:	20	SF:	1.2	brace:	3.745	0.0535
GM:	20	SF:	1.4	brace:	3.77	0.0539
GM:	20	SF:	1.6	brace:	4.315	0.0616
GM:	20	SF:	1.8	brace:	4.868	0.0695
GM:	20	SF:	2	brace:	5.427	0.0775
GM:	20	SF:	2.2	brace:	23.19	0.3313
GM:	20	SF:	2.4	brace:	25.67	0.3667
GM:	20	SF:	2.6	brace:	27.29	0.3898
GM:	20	SF:	2.8	brace:	27.76	0.3965
GM:	20	SF:	3	brace:	27.44	0.392
GM:	20	SF:	3.2	brace:	29.97	0.4281
GM:	20	SF:	3.4	brace:	32.8	0.4686
GM:	20	SF:	3.6	brace:	35.31	0.5044
GM:	20	SF:	3.8	brace:	32.48	0.464
GM:	20	SF:	4	brace:	33.36	0.4765
GM:	20	SF:	4.2	brace:	31.95	0.4564
GM:	20	SF:	4.4	brace:	31.88	0.4554
GM:	20	SF:	4.6	brace:	36.19	0.517
GM:	20	SF:	4.8	brace:	38.11	0.5445
GM:	20	SF:	5	brace:	43.43	0.6205
GM:	20	SF:	5.2	brace:	52.41	0.7487
GM:	20	SF:	5.4	brace:	61.93	0.8847
GM:	20	SF:	5.6	brace:	58.73	0.839
GM:	20	SF:	5.8	brace:	49.1	0.7015
GM:	20	SF:	6	brace:	52.67	0.7524

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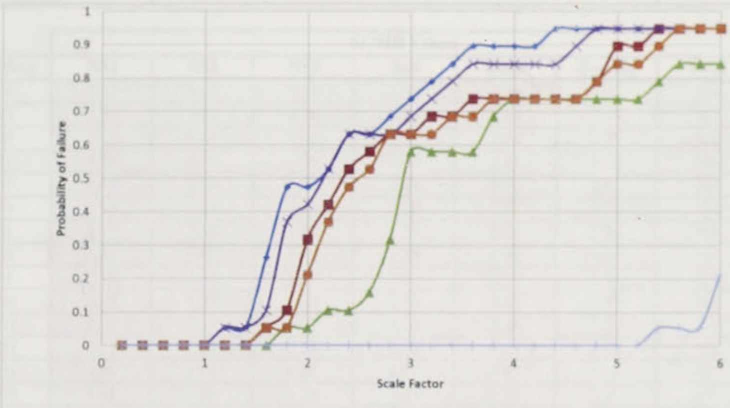
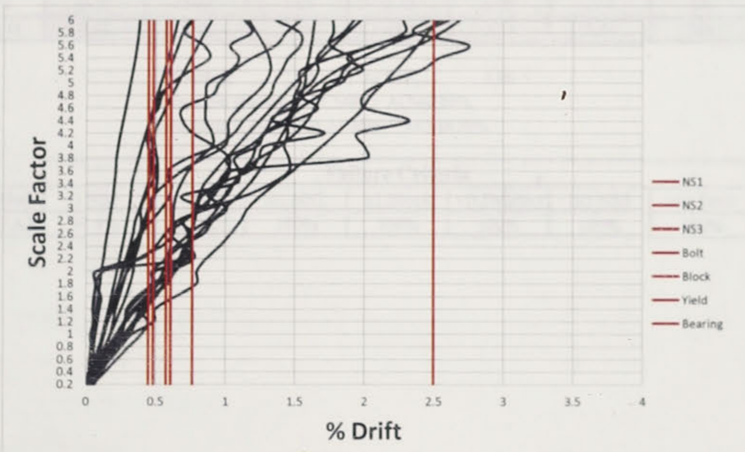
Max Drift (mm)		31.5	40.5	53.7	33.9	42.8	42.7	175.0
Max Drift (%)		0.45	0.58	0.77	0.48	0.61	0.61	2.50

SF	Mean	NS1	NS2	NS3	Bolt	Block	Bearing	Yield
0.2	0.03	0	0	0	0	0	0	0
0.4	0.07	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.6	0.11	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.8	0.15	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.0	0.19	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.2	0.23	0.05	0.00	0.00	0.05	0.00	0.00	0.00
1.4	0.26	0.05	0.00	0.00	0.05	0.00	0.00	0.00
1.6	0.31	0.26	0.05	0.00	0.11	0.05	0.05	0.00
1.8	0.36	0.47	0.11	0.05	0.37	0.05	0.05	0.00
2.0	0.40	0.47	0.32	0.05	0.42	0.21	0.21	0.00
2.2	0.51	0.53	0.42	0.11	0.53	0.37	0.37	0.00
2.4	0.55	0.63	0.53	0.11	0.63	0.47	0.47	0.00
2.6	0.59	0.63	0.58	0.16	0.63	0.53	0.53	0.00
2.8	0.63	0.68	0.63	0.32	0.63	0.63	0.63	0.00
3.0	0.70	0.74	0.63	0.58	0.68	0.63	0.63	0.00
3.2	0.74	0.79	0.68	0.58	0.74	0.63	0.63	0.00
3.4	0.82	0.84	0.68	0.58	0.79	0.68	0.68	0.00
3.6	0.88	0.89	0.74	0.58	0.84	0.68	0.68	0.00
3.8	0.98	0.89	0.74	0.68	0.84	0.74	0.74	0.00
4.0	1.04	0.89	0.74	0.74	0.84	0.74	0.74	0.00
4.2	1.09	0.89	0.74	0.74	0.84	0.74	0.74	0.00
4.4	1.13	0.95	0.74	0.74	0.84	0.74	0.74	0.00
4.6	1.17	0.95	0.74	0.74	0.89	0.74	0.74	0.00
4.8	1.23	0.95	0.79	0.74	0.95	0.79	0.79	0.00
5.0	1.30	0.95	0.89	0.74	0.95	0.84	0.84	0.00
5.2	1.39	0.95	0.89	0.74	0.95	0.84	0.84	0.00
5.4	1.46	0.95	0.95	0.79	0.95	0.89	0.89	0.05
5.6	1.54	0.95	0.95	0.84	0.95	0.95	0.95	0.05
5.8	1.59	0.95	0.95	0.84	0.95	0.95	0.95	0.05
6.0	1.64	0.95	0.95	0.84	0.95	0.95	0.95	0.21
	S_{CT}	2.097	2.546	3.263	2.159	2.715	2.710	
	S_{MT}	1.000	1.000	1.000	1.000	1.000	1.000	1.000
	CMR	2.10	2.55	3.26	2.16	2.71	2.71	0.00
	Δ_u	31.5	40.5	53.7	33.9	42.8	42.7	175.0
	μ_T	1.15	1.48	1.96	1.24	1.56	1.56	6.40
	SSF	1.03	1.059	1.078	1.041	1.063	1.062	1.172
	ACMR	2.17	2.69	3.52	2.25	2.88	2.88	0.00

	CALCULATION SHEET	PROJECT NO.: M.ENG
CLIENT: MCGILL	PREPARED: A.G.	PAGE OF
PROJECT: CBFs Building 7H	CODE: NA	13 13
SUBJECT: IDA Results and Analysis	DATE: 2012-12-01	





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CLIENT: MCGILL	PREPARED: A.G.	PAGE	OF
PROJEC CBFs	CODE: NA	1	1
SUBJECT: Summary of IDA Results	DATE: 2012-12-01		

	ACMR Values						
Building	**NS1**	**NS2**	**NS3**	**Bolt**	**Block**	**Bearing**	**Drift**
1H	2.41	3.12	4.25	2.92	3.32	3.60	NA
2H	2.29	3.03	4.18	2.73	3.16	3.47	NA
3H	2.40	3.05	3.92	2.75	3.14	3.25	NA
4H	2.38	3.29	3.96	2.60	3.44	3.43	NA
5H	1.98	2.55	3.32	2.21	2.76	2.85	NA
6H	1.93	2.55	3.27	2.08	2.77	2.75	NA
7H	2.17	2.69	3.52	2.25	2.88	2.88	NA
8H	3.16	3.83	5.12	3.35	3.97	3.95	NA
9H	2.80	3.57	4.40	3.03	3.72	3.74	NA
10H	2.54	3.27	4.08	2.73	3.42	3.41	NA
11H	2.41	3.17	3.92	2.56	3.38	3.30	NA
12H	2.23	2.83	3.56	2.30	3.05	2.86	NA
13H	2.56	3.03	3.60	2.67	3.13	3.11	NA
14H	2.34	2.82	3.28	2.43	2.93	2.86	NA
15H	2.88	3.49	4.29	2.93	3.65	3.50	NA
16H	2.22	2.92	3.63	2.20	3.09	2.82	NA
Avg H	2.42	3.08	3.89	2.61	3.24	3.24	NA

$ACMR_{10\%}$ 2.79 $ACMR_{20\%}$ 1.96

Acceptable if average ACMRi > ACMR10%

Acceptable if individual ACMRi > ACMR20%

	Failure Criteria						
Location	i) NS1	iv) Bolt	ii) NS2	v) Block	vi) Bearing	iii) NS3	vii) Drift
HALIFAX	93.8%	100%	100%	100%	100%	100%	100%

Appendix C

Sample calculation of building 7H for FEMA P-695 acceptance criteria.

For Building 7H:

- Ground motion : 1
- Scaling factor : 1
- Failure criteria: NS1

From testing by Caruso-Juliano (2012):

Failure	δ_u (mm)	P (kN)	L_{tot} (mm)	L_{cnx} (mm)	A_{test} (mm^2)
NS 1 (3Bsa)	12.1	402.0	3070	228.6	1362

where:

δ_u = ultimate elongation of brace tested;

P = axial test load;

L_{tot} = total length of brace tested;

L_{cnx} = length of connection of brace tested;

A_{test} = area of brace tested.

Removing elastic portion of elongation, $\delta_{elastic}$, from the ultimate elongation, δ_u:

$$(\delta_u - \delta_{elastic})_{test} = \frac{P(L_{tot} - L_{cnx})}{EA} = 7.91 \text{ mm}$$

Applying test results to building 7 (Layout Bc) with:

Structure Type	L_{brace} (mm)	A_{brace} (mm^2)	Width Bay, W (mm)	Height, H (mm)
7	10259	768	7500	7000

Adjusting for length of brace for building 7H versus test results, the ultimate elongation for the braced bay in building type 7, $(\delta_u)_{BLD\,7}$ becomes:

$$(\delta_u)_{BLD\,7} = (\delta_u - \delta_{elastic})_{test} + \frac{PL_{brace}}{EA_{test}} = 23.0 \text{ mm}$$

Such that the percent elongation at failure is:

$$\%\,(\delta_u)_{BLD\,7} = \frac{(\delta_u)_{BLD\,7}}{L_{brace}}\,x\,100 = 0.22\%$$

Maximum building drift at failure:

$$(\Delta_u)_{BLD\,7} = \left\{\sqrt{[(L_{brace} + (\delta_u)_{BLD\,7})^2 - H^2]} - W\right\} = 31.5 \text{ mm}$$

Such that the percent drift at failure is:

$$\%\,\Delta_u = \frac{(\Delta_u)_{BLD\,7}}{H}\,x\,100 = 0.45\%$$

Then from IDA analysis for ground motion 1 and scaling factor 1.0:

$$\Delta_{brace} = 8.499 \text{ mm}$$

This corresponds to a percent drift of:

$$\%\,\Delta_{brace} = \frac{\Delta_{brace}}{H}\,x\,100 = 0.12\%$$

The percent drift is averaged for the 20 ground motions, for every scaling factor. For scaling factor 1:

$$\left(\sum_{GM\,1}^{GM\,20} \%\,\Delta_{brace}\right)\Big/20 = 0.19\%$$

The percent drift at failure of 0.45% is therefore seen to occur between scaling factors 2.0 and 2.2, which have average drifts of 0.40% and 0.51% respectively. Therefore the median collapse intensity, $\hat{S}_{CT}$, is then interpolated for the percent drift at failure between these scaling factors :

$$\hat{S}_{CT} = \frac{2.2 - 2}{0.51 - 0.40}(0.45 - 0.40) + 2 = 2.1$$

Since the earthquake records used in the study are based on matching the uniform hazard spectrum the maximum considered earthquake spectral demand, S_{MT}, is taken as 1.0. As such the collapse margin ratio (CMR) is equal to the median collapse intensity, $\hat{S}_{CT}$.

$$\text{CMR}=\frac{\hat{S}_{CT}}{S_{MT}} = \hat{S}_{CT} = 2.1$$

The period-based ductility, μ_T, is taken as the ratio of ultimate roof drift, Δ_u, to effective yield drift, Δ_y. The drift at yield is calculated based on the elongation at yield, δ_y, and the yield load, P_y:

$$\delta_y = \frac{P_y L_{brace}}{E A_{brace}} = 20.0\ mm$$

$$\Delta_y = \left\{\sqrt{\left[\left(L_{brace} + \delta_y\right)^2 - H^2\right]} - W\right\} = 27.3\ mm$$

$$\mu_T = \Delta_u/\Delta_y = 1.15$$

The spectral shape factor, SSF, varies according to the fundamental period, T, and period-based ductility; μ_T. For a period of 0.779 for building 7H, SSF is interpolated as1.03.

The adjusted collapse margin ratio (ACMR) is then calculated for failure criteria NS1:

$$\text{ACMR}_{\text{NS1}} = \text{SSF}_{\text{NS1}} \times \text{CMR}_{NS1} = 2.17$$

Total system collapse uncertainty, β_{TOT}, is calculated as:

$$\beta_{TOT} = \sqrt{B^2{}_{RTR} + B^2{}_{DR} + B^2{}_{TD} + B^2{}_{MDL}} = 0.80$$

where:

$$\beta_{RTR} = 0.40;\ \beta_{DR} = 0.30;\ \beta_{TD} = 0.45;\ \beta_{MDL} = 0.45$$

From Table 7-3 of FEMA-P695 the acceptable adjusted collapse margin ratio, $\text{ACMR}_{10\%}$, for each performance group is taken as:

$$\mathrm{ACMR}_{10\%} = 2.79$$

The acceptable adjusted collapse margin ratio for a specific archetype, $\mathrm{ACMR}_{20\%}$, is taken as:

$$\mathrm{ACMR}_{20\%} = 1.96$$

For each performance group, the acceptable adjusted collapse margin ratio, $\mathrm{ACMR}_{10\%}$, is compared to the average adjusted collapse margin ratio, ACMR, of all archetypes.

For NS1, the average adjusted collapse margin ratio, ACMR is:

$$\mathrm{ACMR}_{\mathrm{NS1}} = 2.42 < ACMR_{10\%}$$

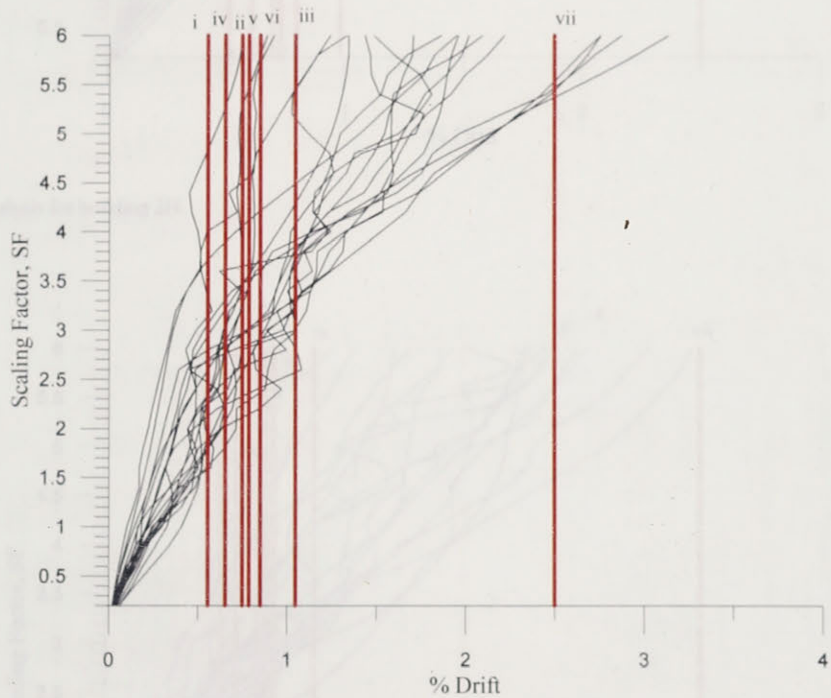
Within each performance group, the adjusted collapse margin ratio, ACMR, of each archetype individually is compared to the acceptable adjusted collapse margin ratio relaxed to a probability of collapse of 20%, $\mathrm{ACMR}_{20\%}$. For the critical case, 6H:

$$\mathrm{ACMR}_{\mathrm{6H}} = 1.96 < ACMR_{20\%}$$

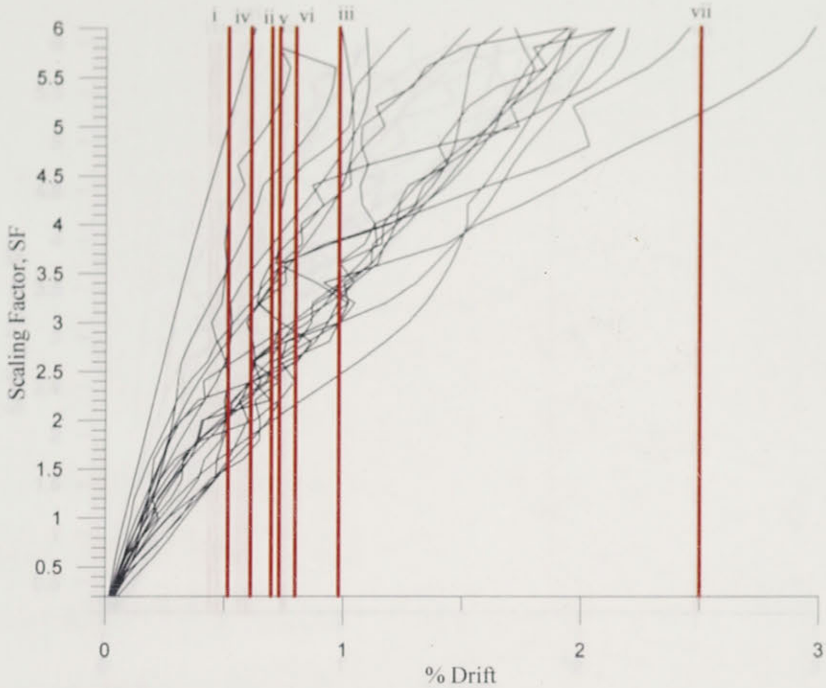
NS1 does not meet the performance criteria.

Appendix D

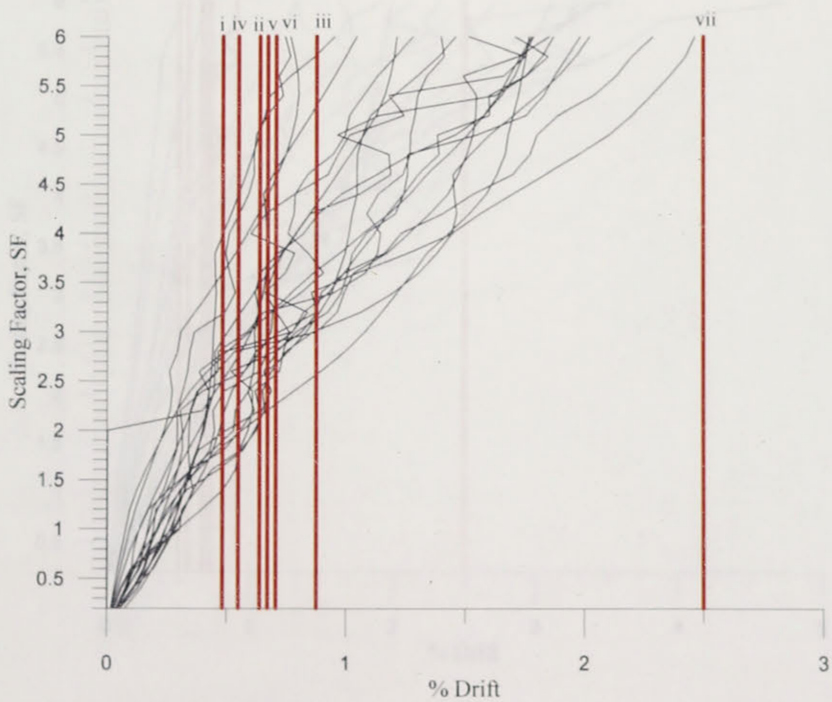
	Failure Criteria
i	NS 1 (3Bsa)
ii	NS 2 (3Cs)
iii	NS 3 (3As)
iv	Bolt Shear (D05X)
v	Block Shear (D06X)
vi	Bearing (D03X)
vii	Yield (drift)



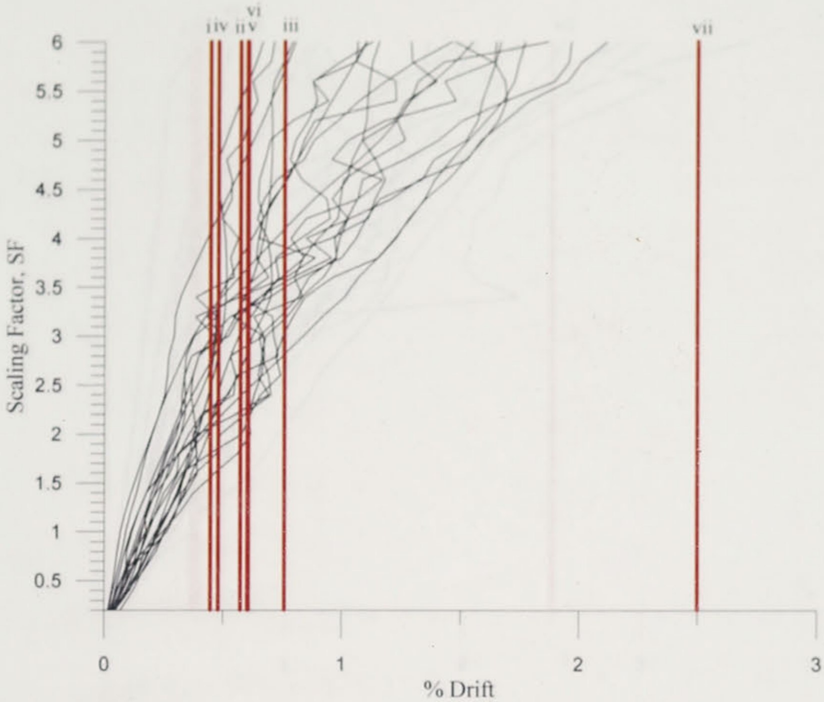
Results of IDA analysis for building 1H.



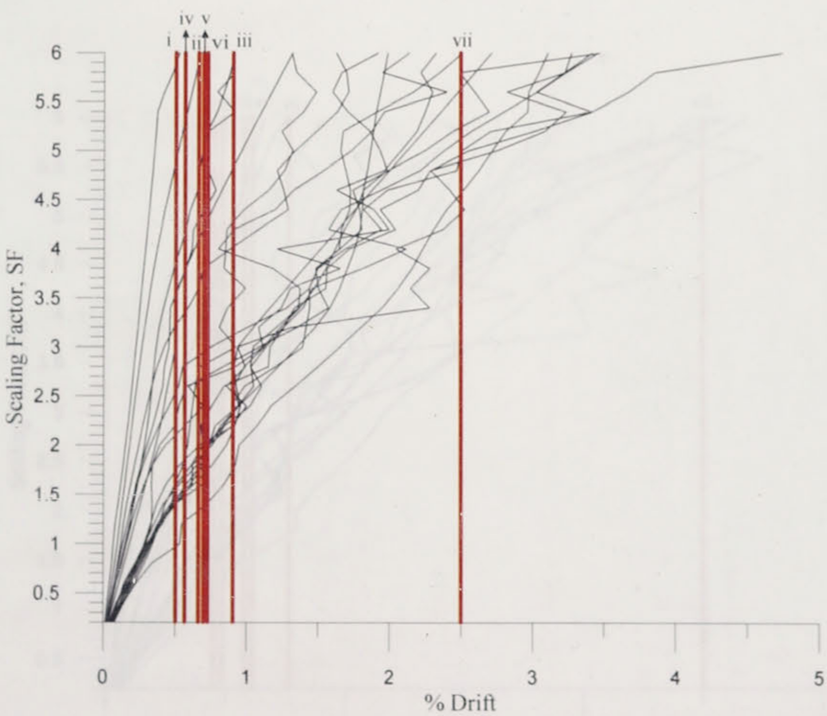
Results of IDA analysis for building 2H.



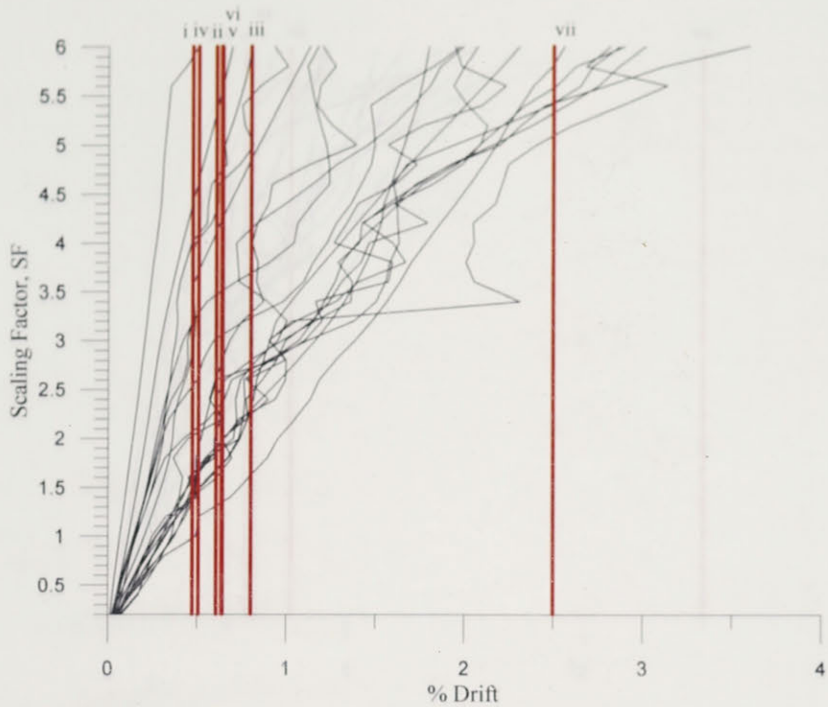
Results of IDA analysis for building 3H.



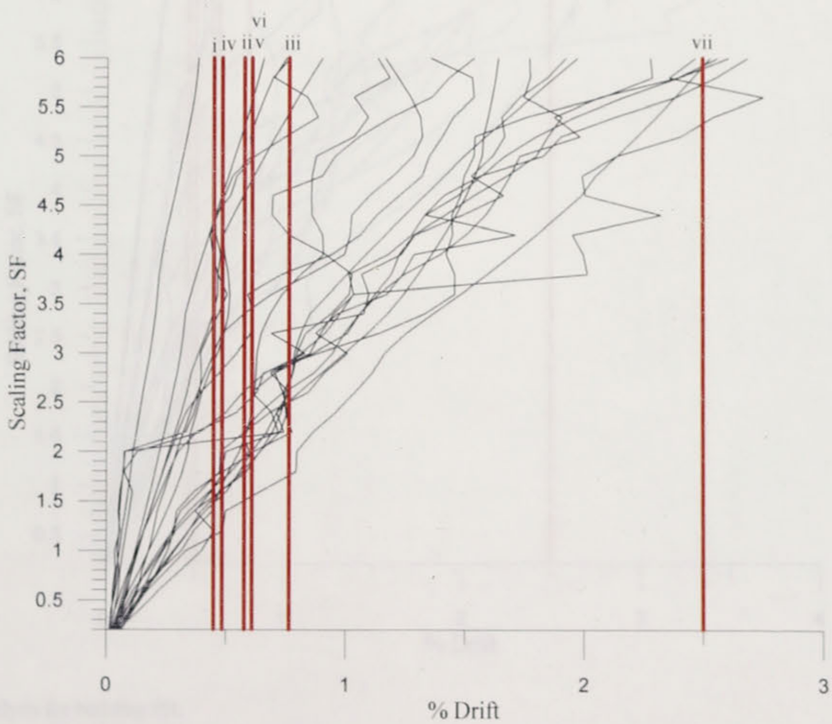
Results of IDA analysis for building 4H.



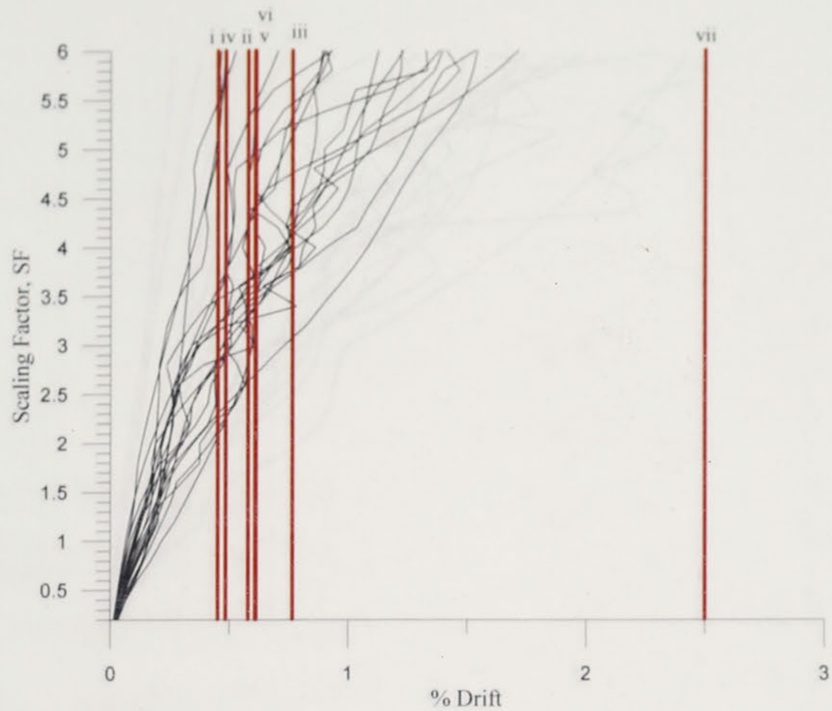
Results of IDA analysis for building 5H.



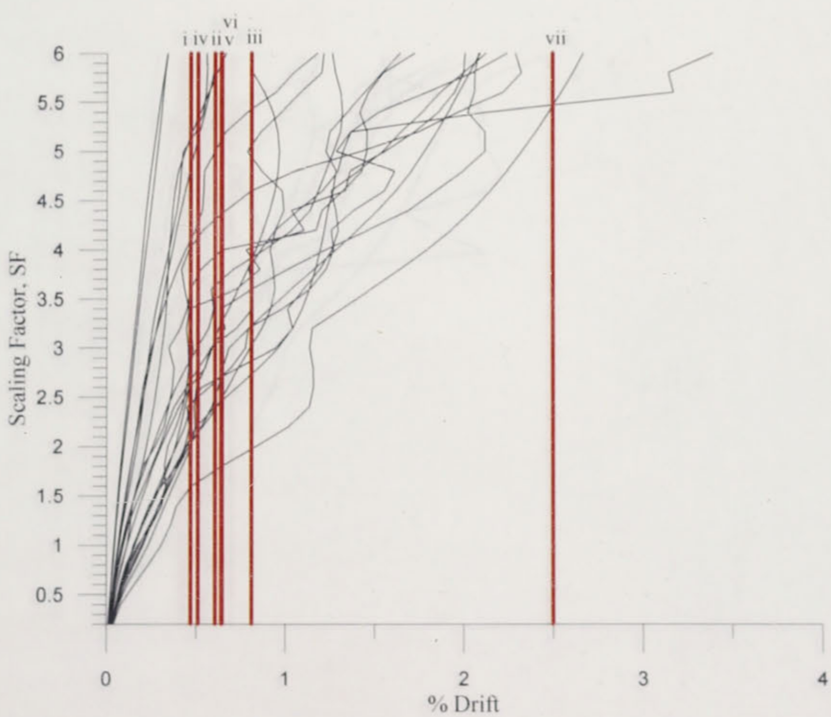
Results of IDA analysis for building 6H.



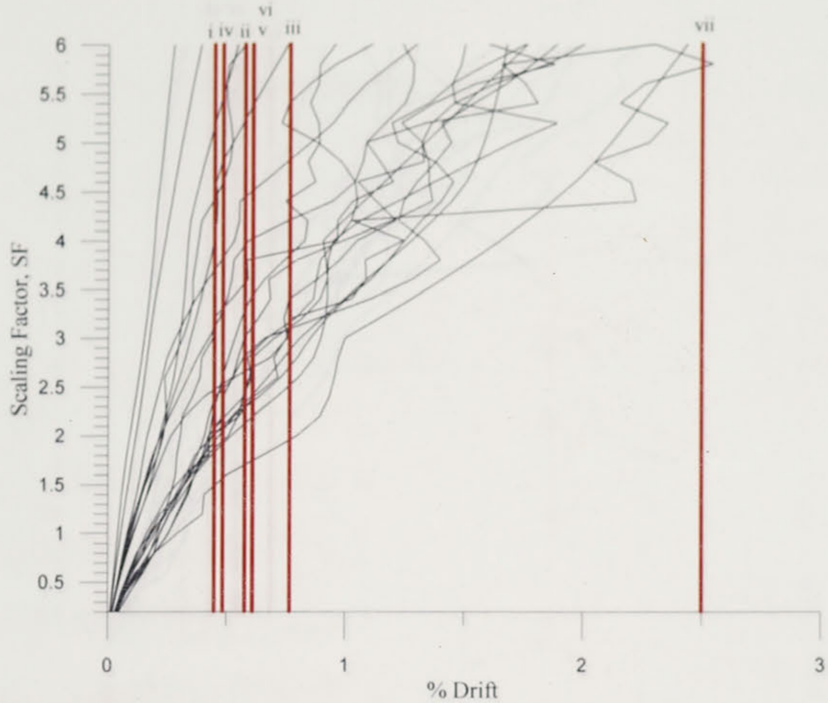
Results of IDA analysis for building 7H.



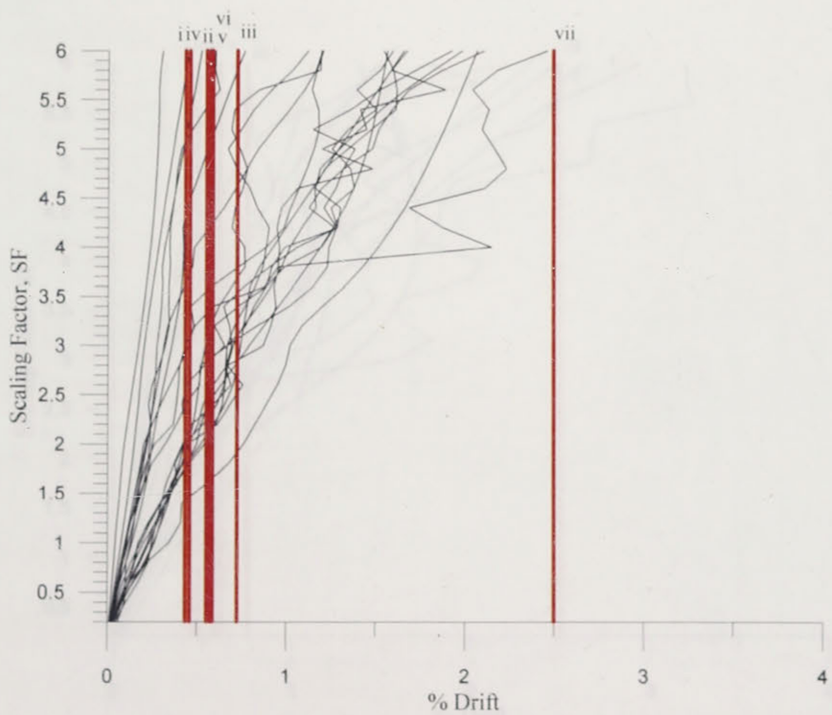
Results of IDA analysis for building 8H.



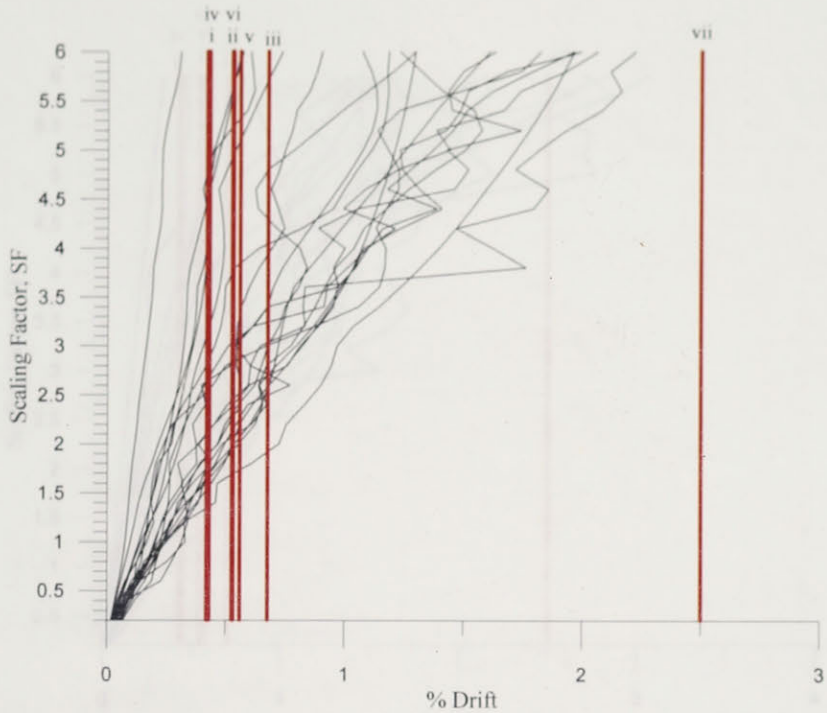
Results of IDA analysis for building 9H.



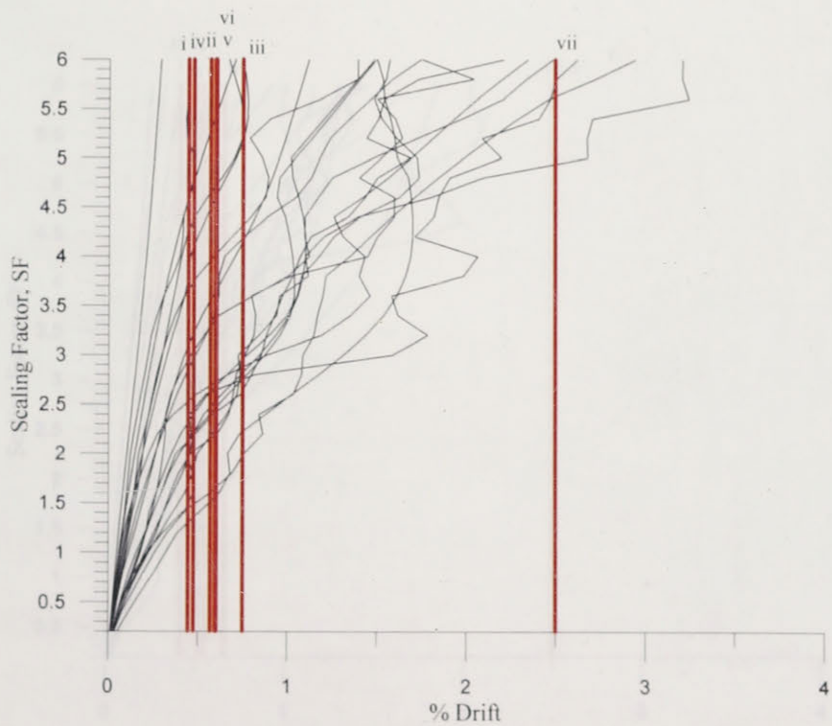
Results of IDA analysis for building 10H.



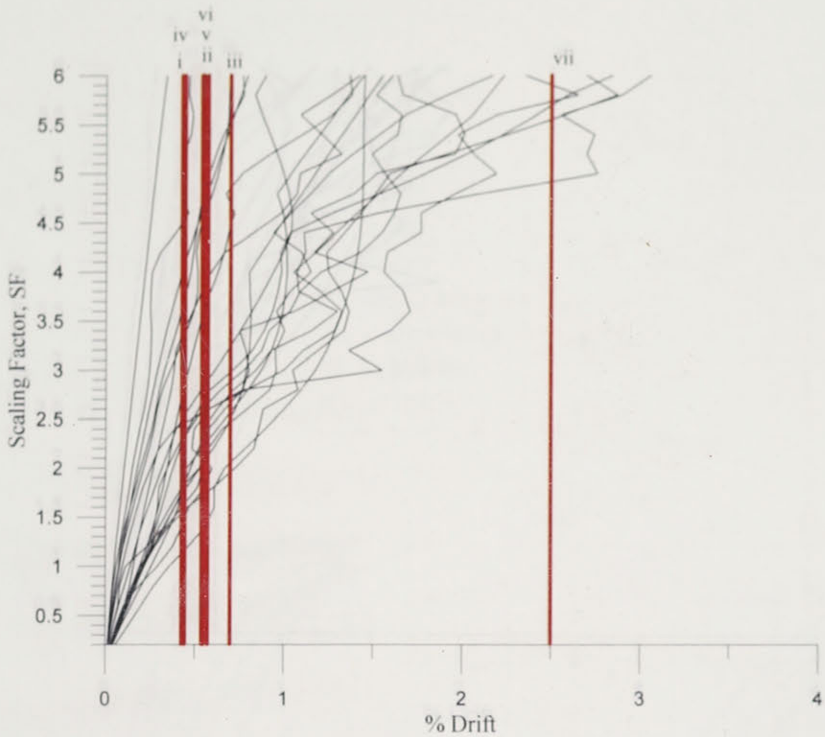
Results of IDA analysis for building 11H.



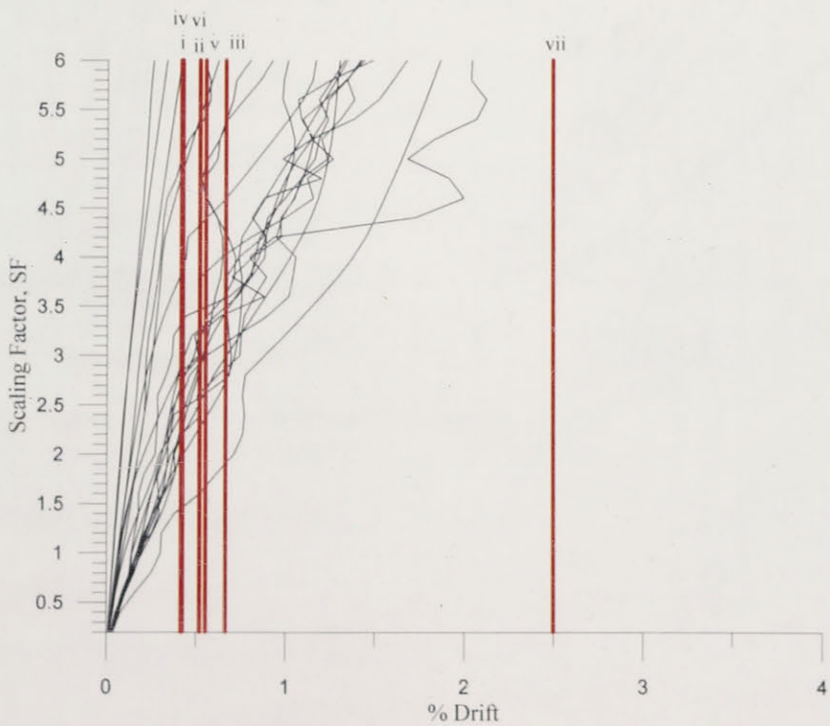
Results of IDA analysis for building 12H.



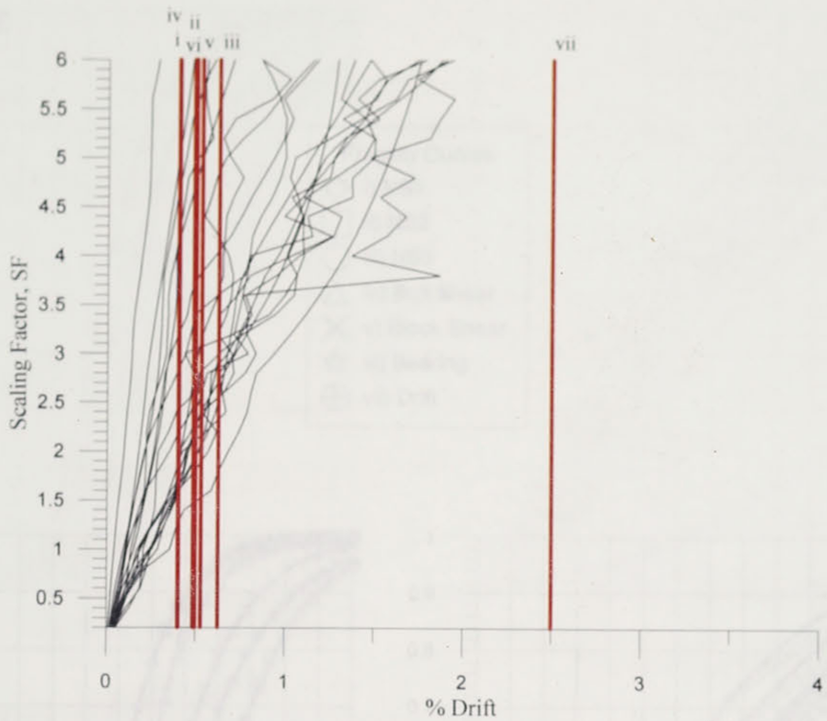
Results of IDA analysis for building 13H.



Results of IDA analysis for building 14H.

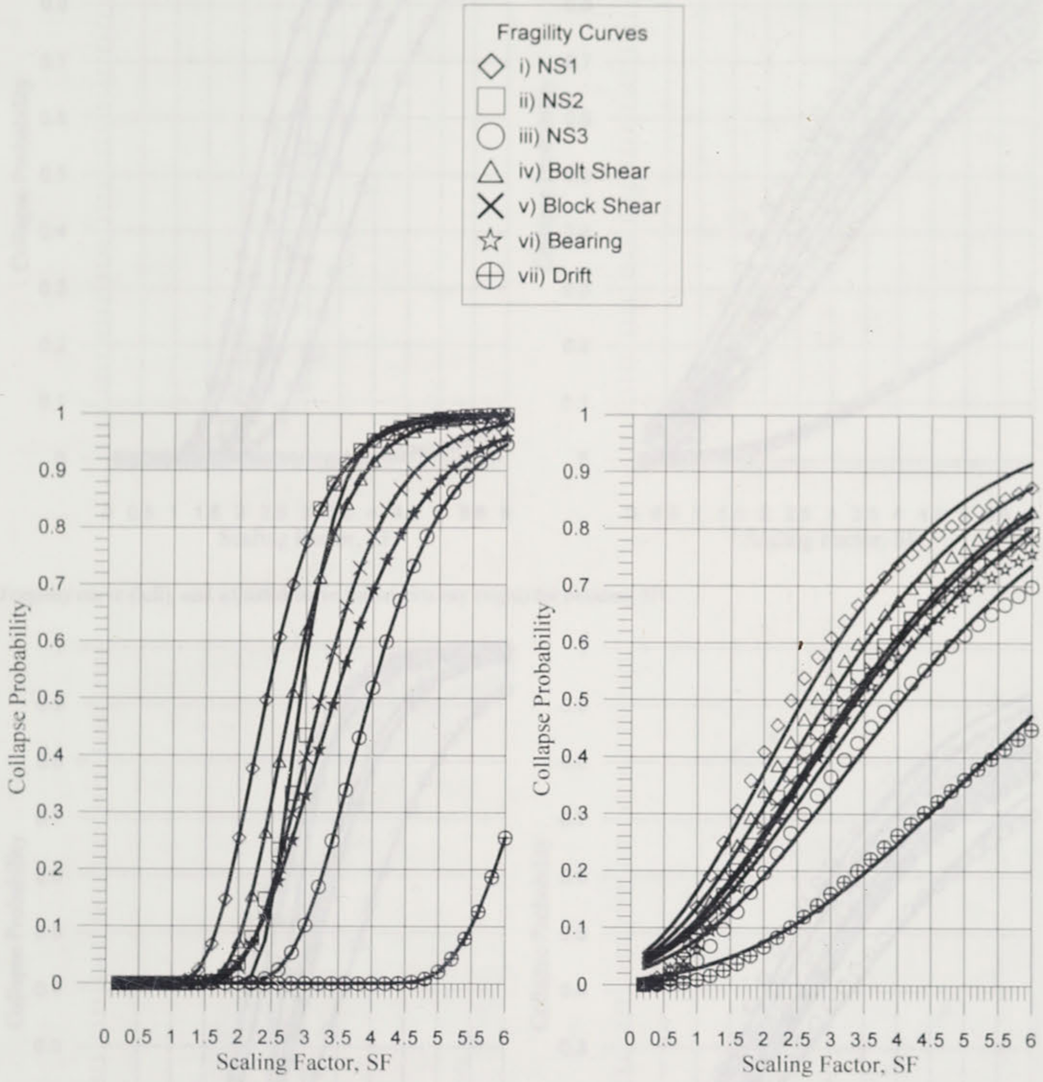


Results of IDA analysis for building 15H.

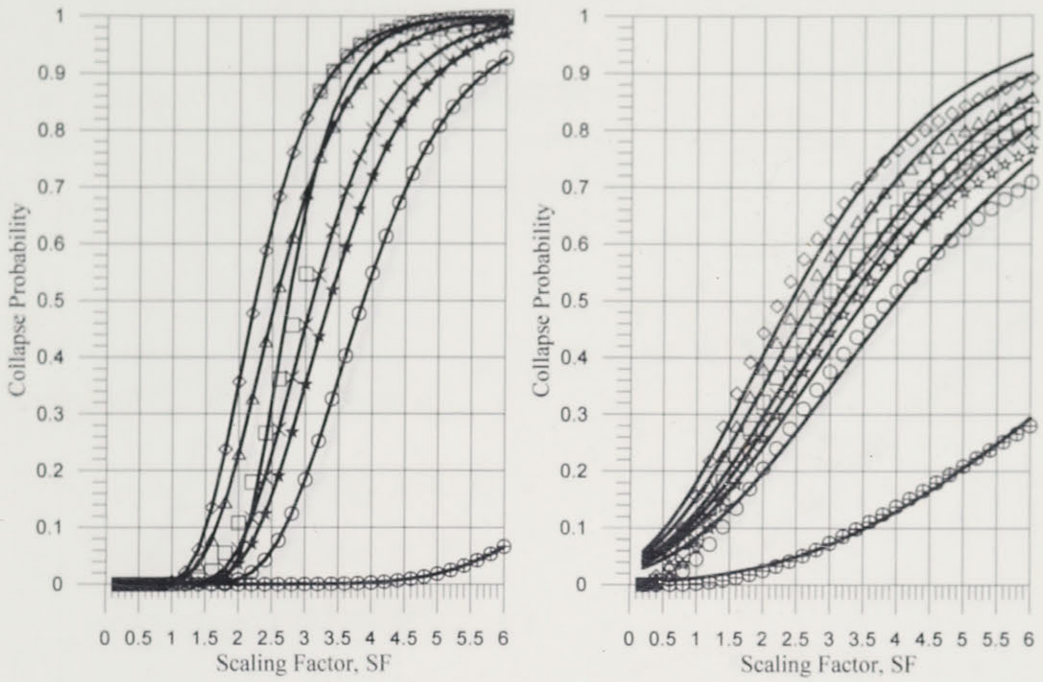


Results of IDA analysis for building 16H.

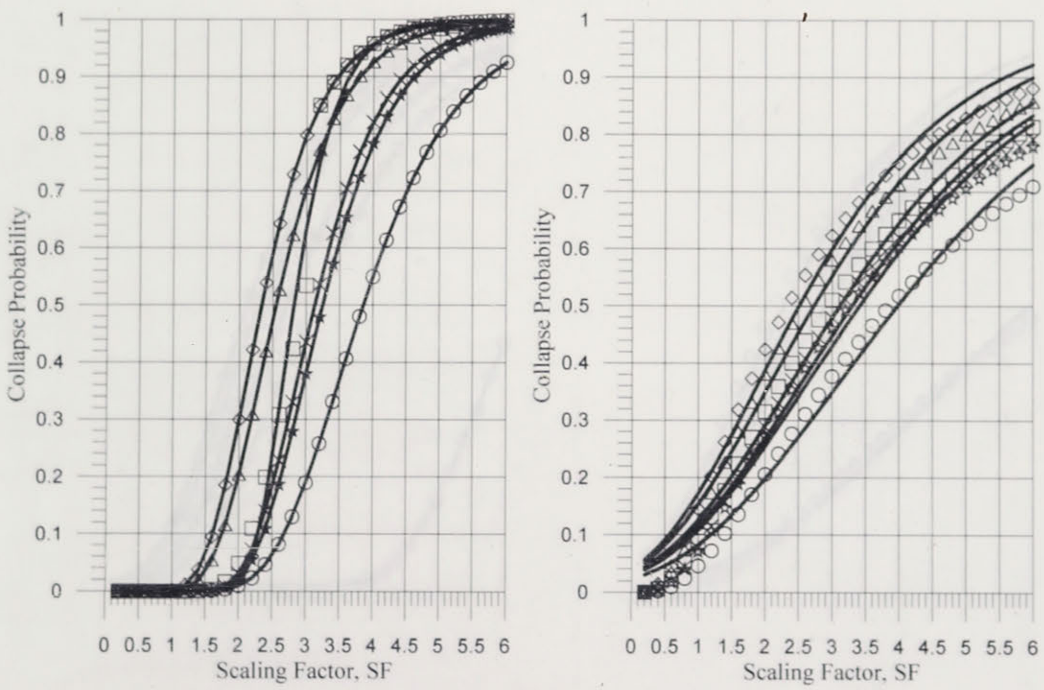
Appendix E



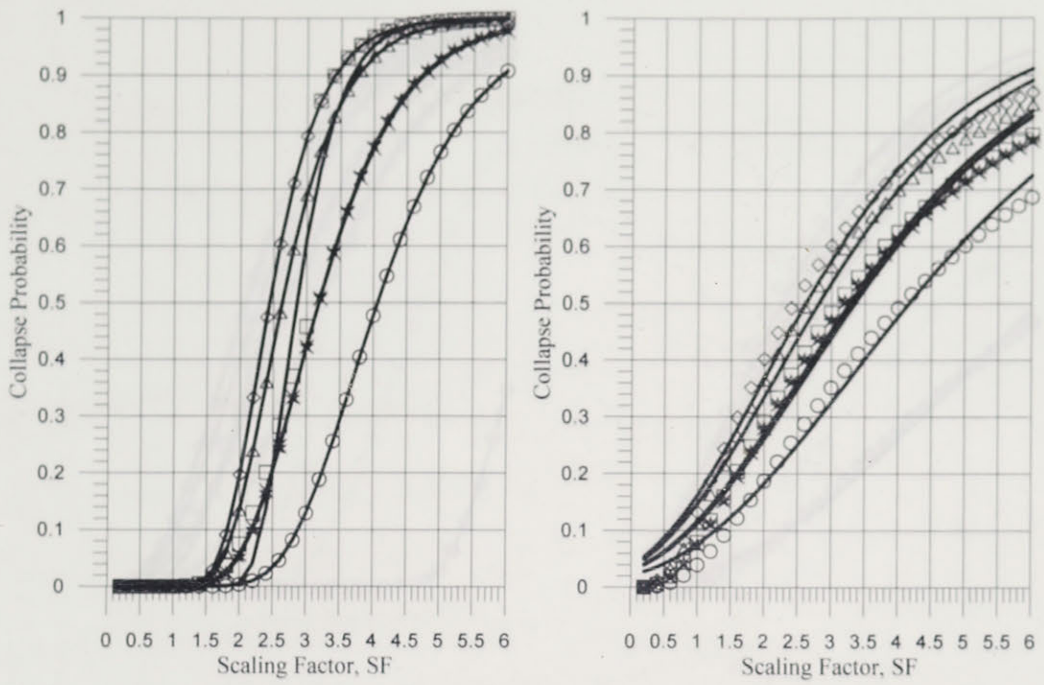
Fragility curve (left), and adjusted curve for uncertainty (right) for building 1H.



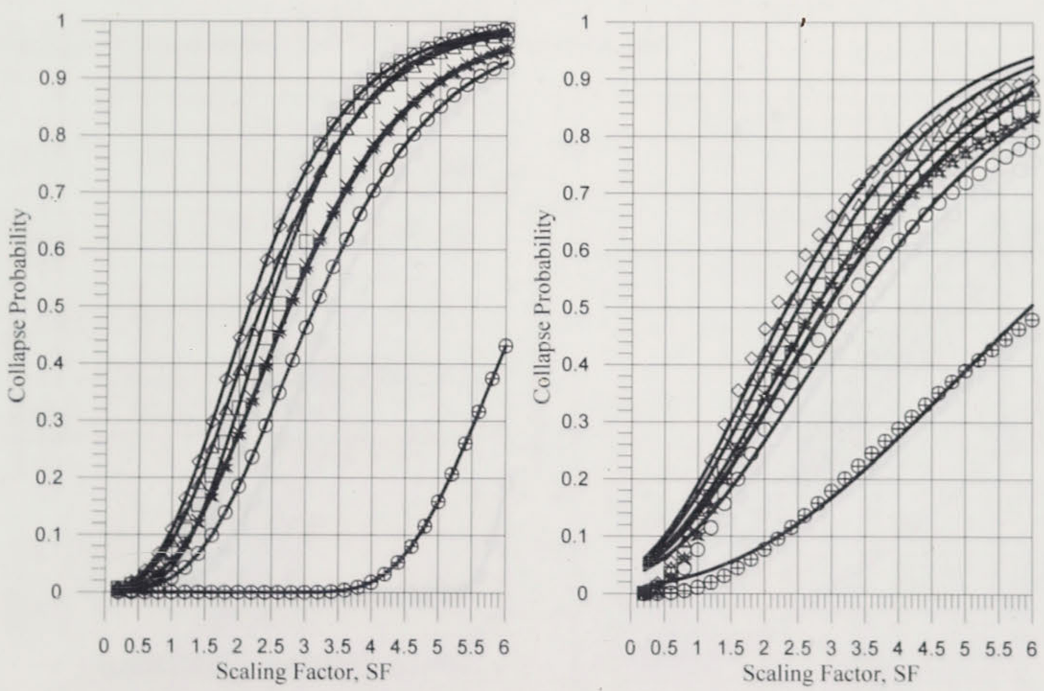
Fragility curve (left), and adjusted curve for uncertainty (right) for building 2H.



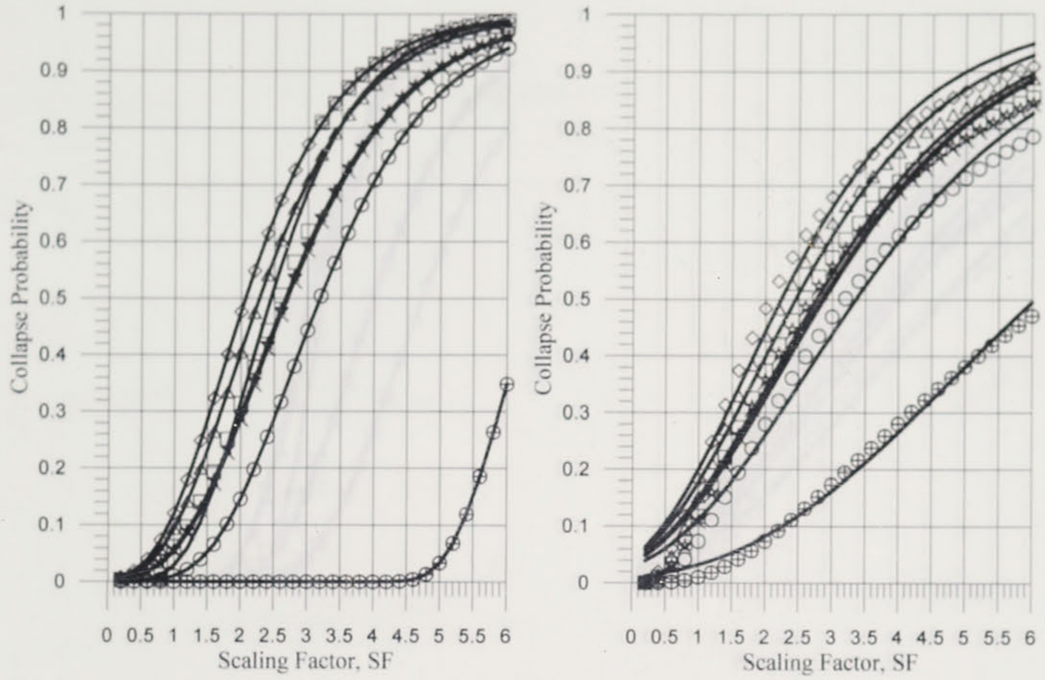
Fragility curve (left), and adjusted curve for uncertainty (right) for building 3H.



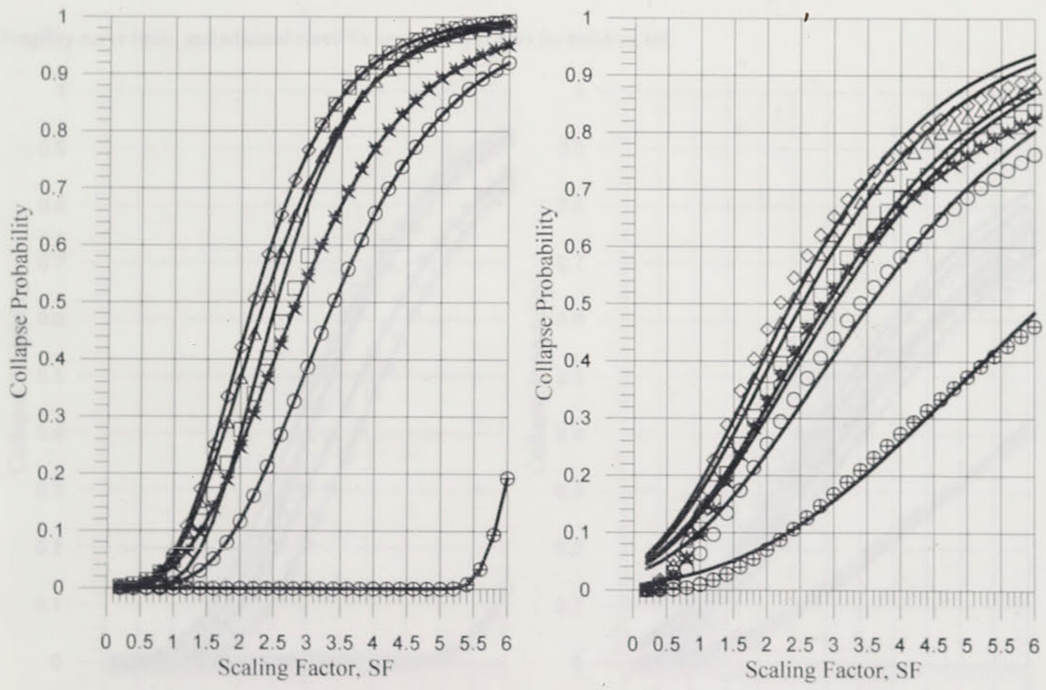
Fragility curve (left), and adjusted curve for uncertainty (right) for building 4H.



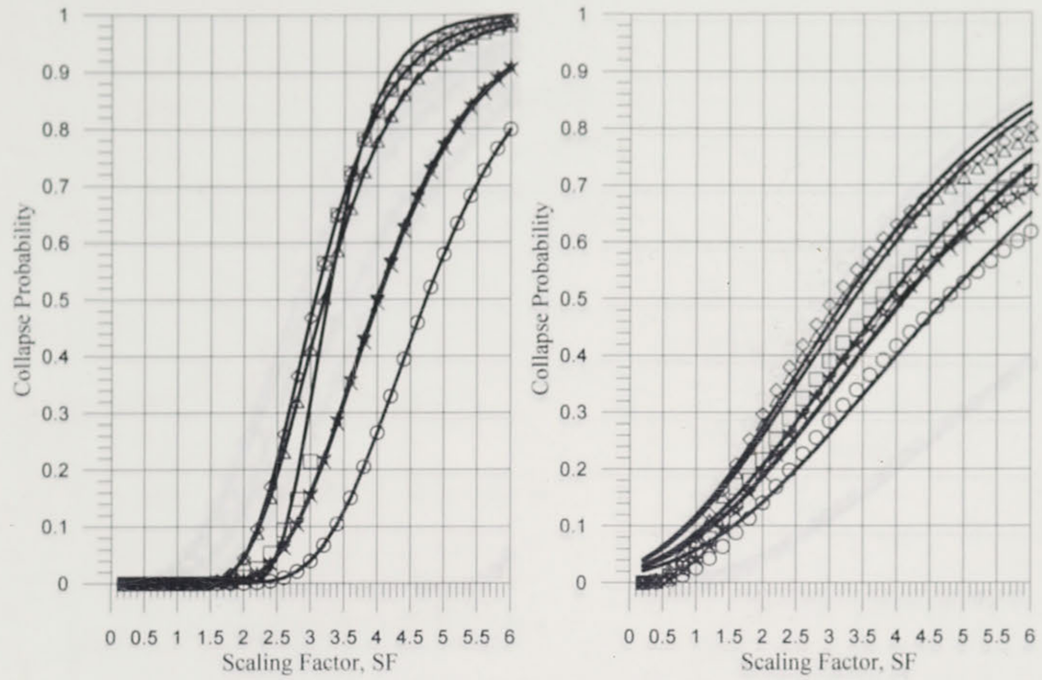
Fragility curve (left), and adjusted curve for uncertainty (right) for building 5H.



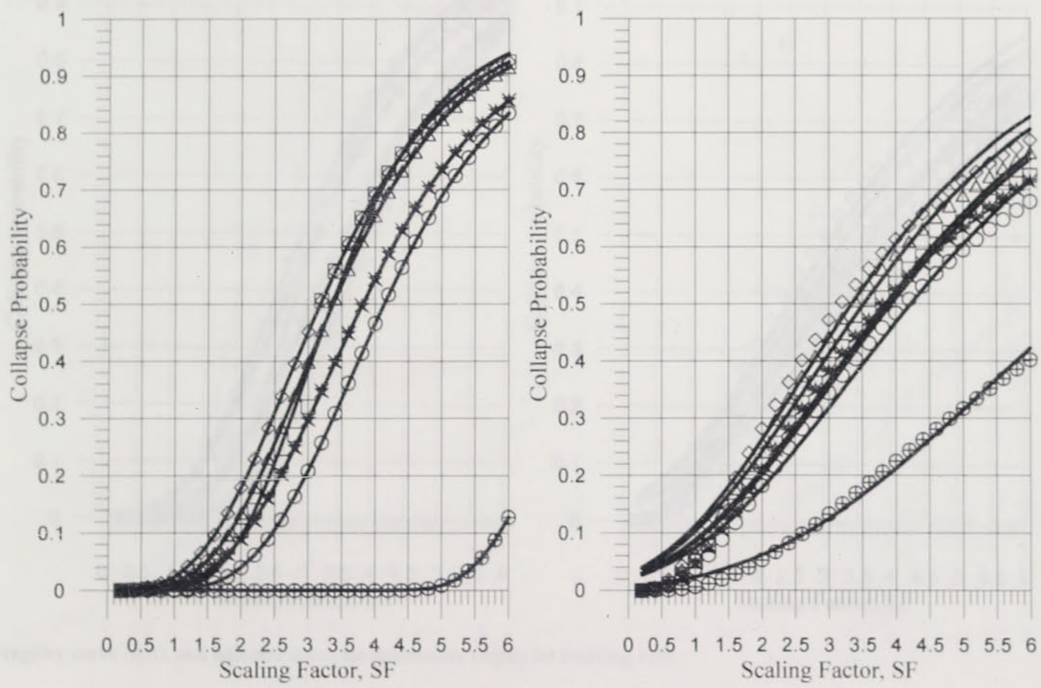
Fragility curve (left), and adjusted curve for uncertainty (right) for building 6H.



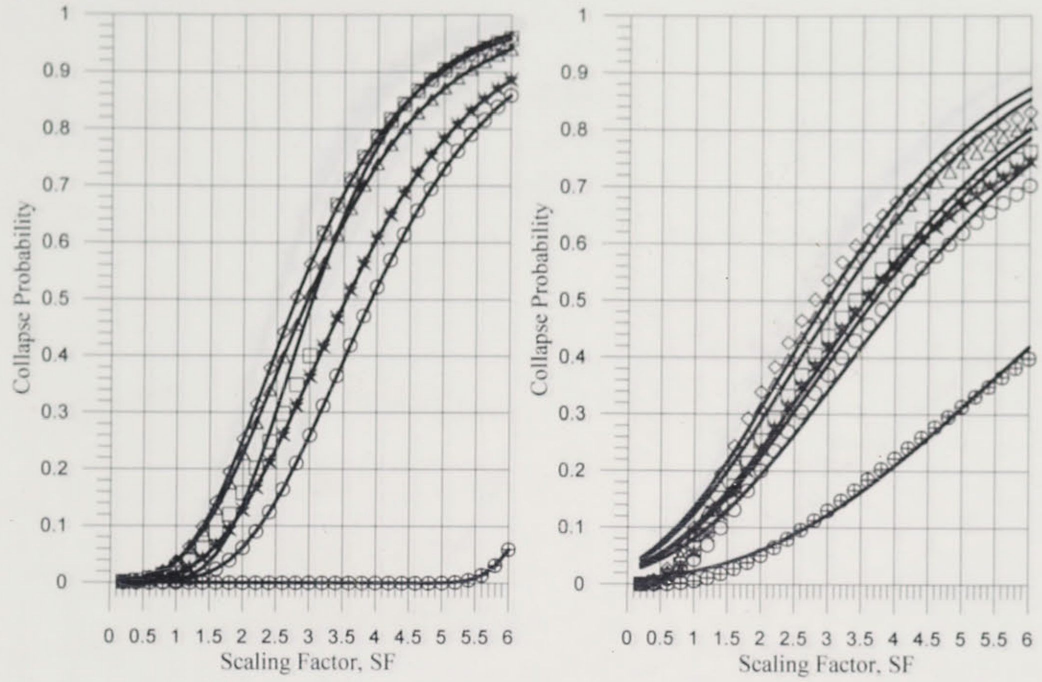
Fragility curve (left), and adjusted curve for uncertainty (right) for building 7H.



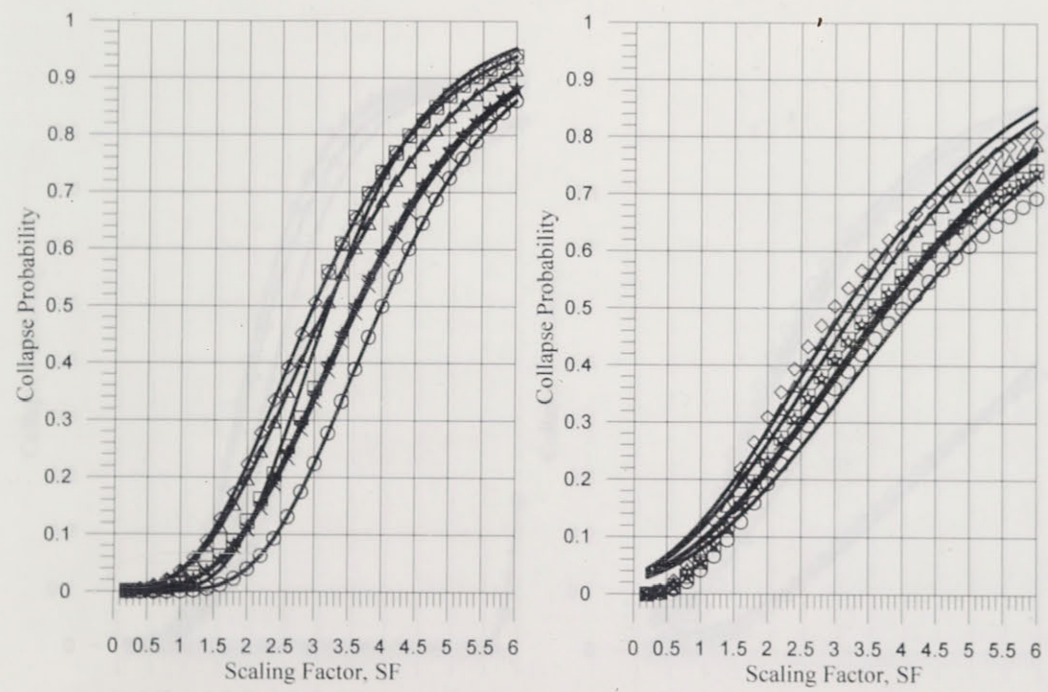
Fragility curve (left), and adjusted curve for uncertainty (right) for building 8H.



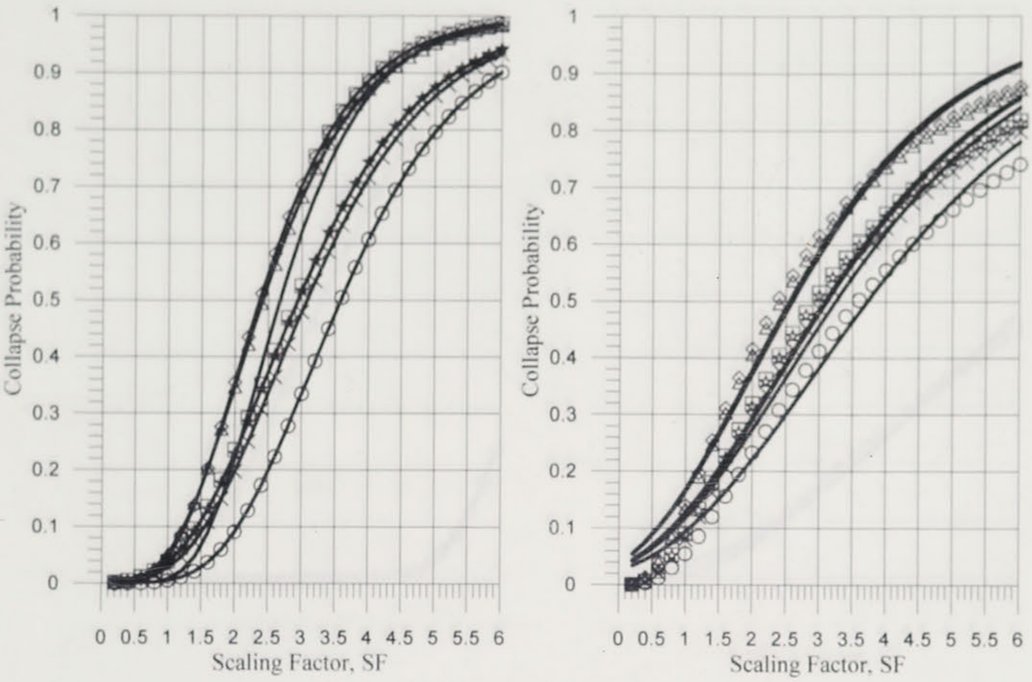
Fragility curve (left), and adjusted curve for uncertainty (right) for building 9H.



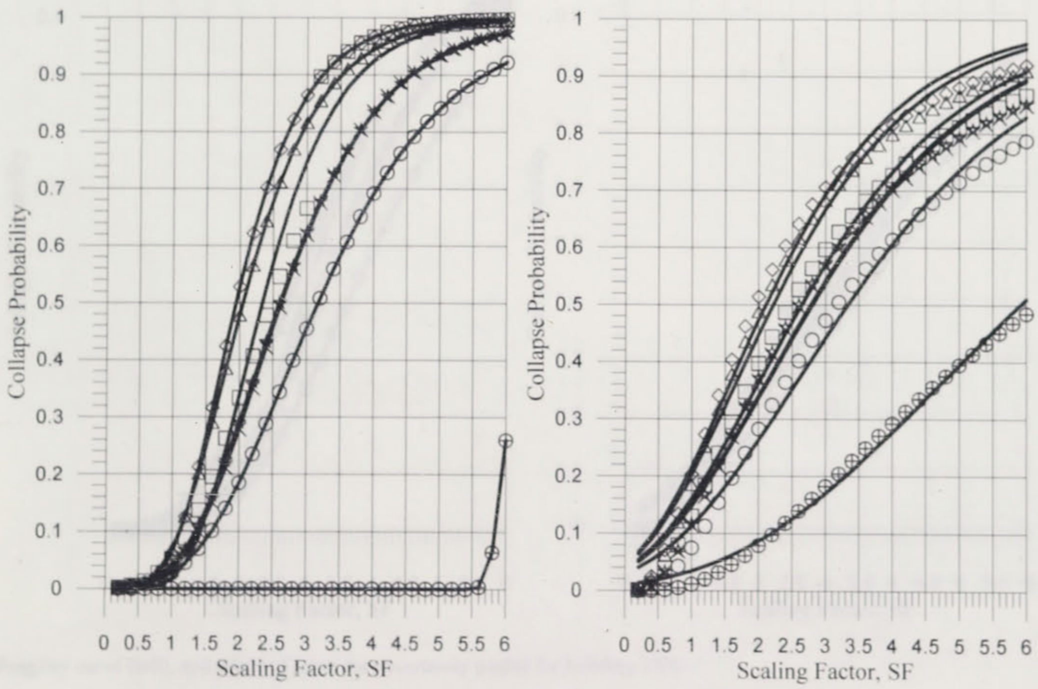
Fragility curve (left), and adjusted curve for uncertainty (right) for building 10H.



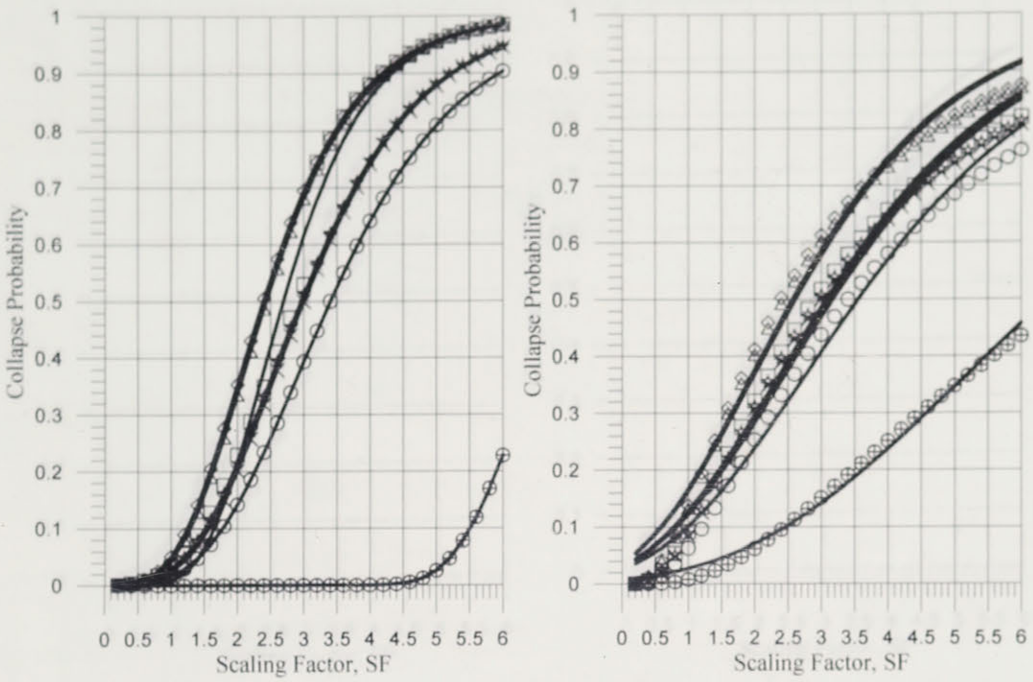
Fragility curve (left), and adjusted curve for uncertainty (right) for building 11H.



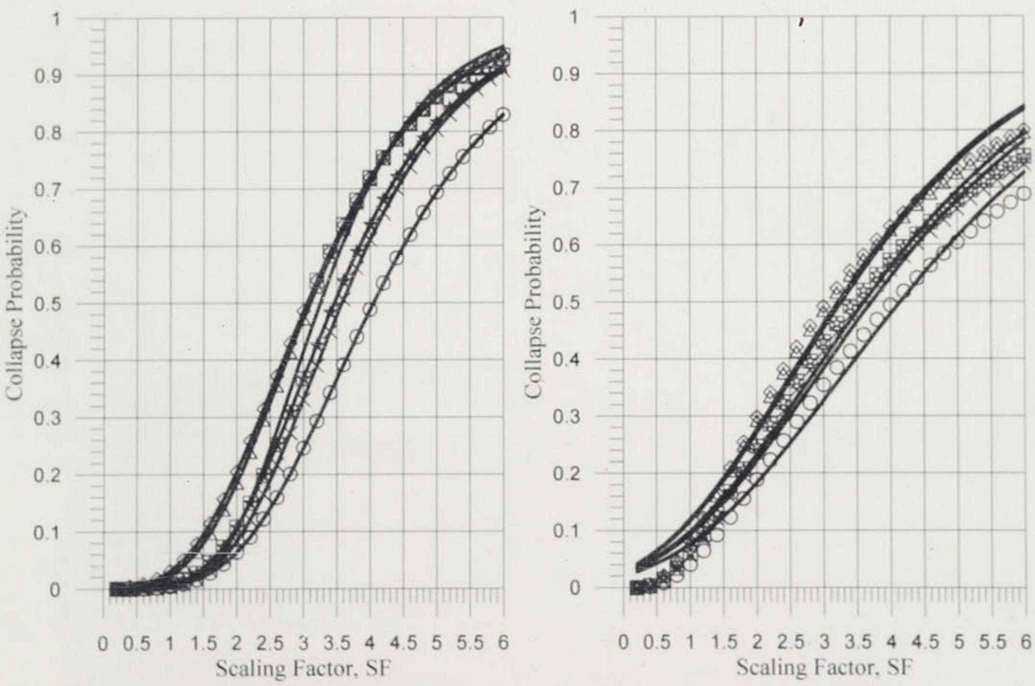
Fragility curve (left), and adjusted curve for uncertainty (right) for building 12H.



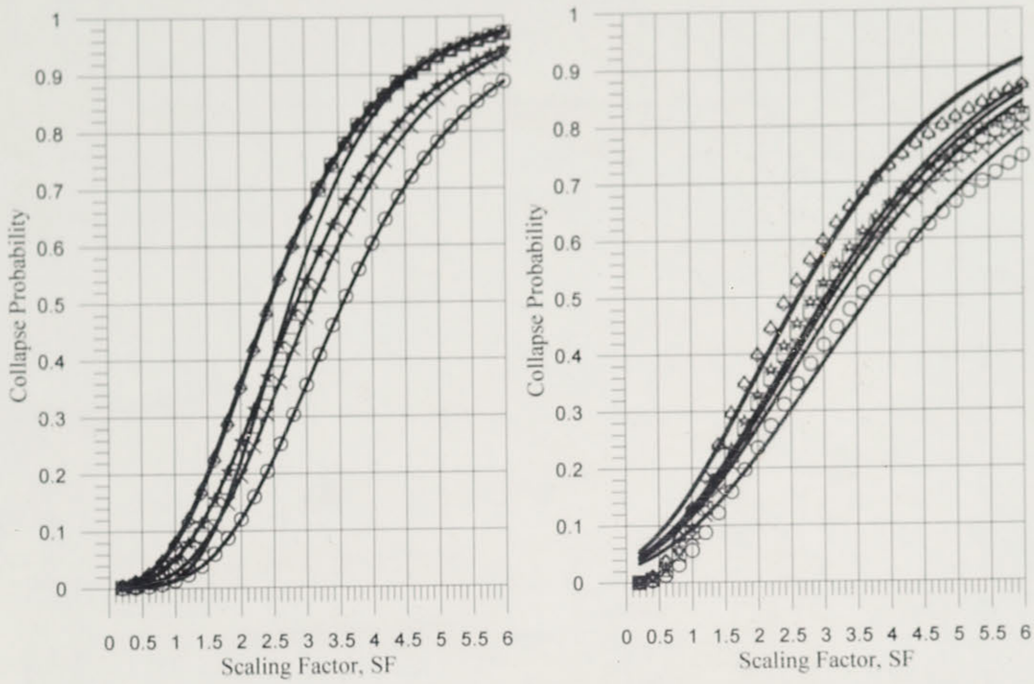
Fragility curve (left), and adjusted curve for uncertainty (right) for building 13H.



Fragility curve (left), and adjusted curve for uncertainty (right) for building 14H.



Fragility curve (left), and adjusted curve for uncertainty (right) for building 15H.



Fragility curve (left), and adjusted curve for uncertainty (right) for building 16H.