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Comparison of US and Canadian code requirements for seismic design of steel buildings

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Abstract

The US system of building codes, which incorporate the AISC Seismic Provisions, have increasingly looked to their Canadian counterparts for guidance on a more thorough approach to expected mechanism formation in lateral systems. Recent changes in the AISC Seismic Provisions require explicit consideration of a plastic mechanism in the design of ductile braced-frame systems. Lateral elements other than the intended fuse now must be designed to remain elastic even as the fuse elements reach their expected strength. This paper highlights design and analysis requirement changes between the 2005 and 2010 editions of the AISC Seismic Provisions (AISC, 2005; AISC, 2010) and compares them to those in the CAN/CSA-S16-01 and S16S1-05 Supplement (CSA, 2005), adopted by the 2005 edition of the National Building Code of Canada (NRCC/CNRC, 2005). Comparison designs are performed for comparable regions of high seismicity (the Seattle/Vancouver area) and the performance of the buildings is evaluated and discussed.

I. Introduction

This paper highlights the differences in seismic design provisions for the design of braced frames between the 2005 and 2010 editions of AISC 341. The changes bring the AISC code closer to the NBCC/CSA 2005 Canadian seismic design provisions. AISC 341 2010 and NBCC/CSA 2005 both require explicit consideration of plastic mechanism(s) to ensure that the intended sources of ductility (braces) behave as structural fuses while the associated beams and columns remain essentially elastic.

II. Building Description

The structure to be analyzed is a four-story office building with two identical braced bays at each line of perimeter framing. The floorplate is square with 121.3 ft (37.0 m) sides. Story heights are 15 ft (4.6 m) at the ground floor and reduce to 12 ft (3.7m) above, bringing the total building height to 51 ft (15.5 m). An elevation of the braced frames used in subsequent comparisons is shown in Figure 1, while a plan of the floor framing can be seen in Figure 2.

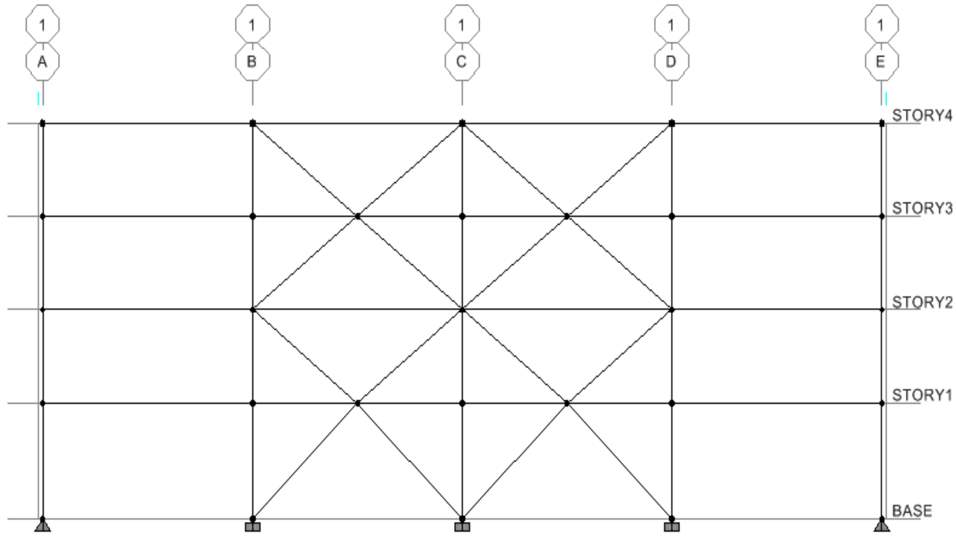

Figure 1: Typical Braced Frame Elevation

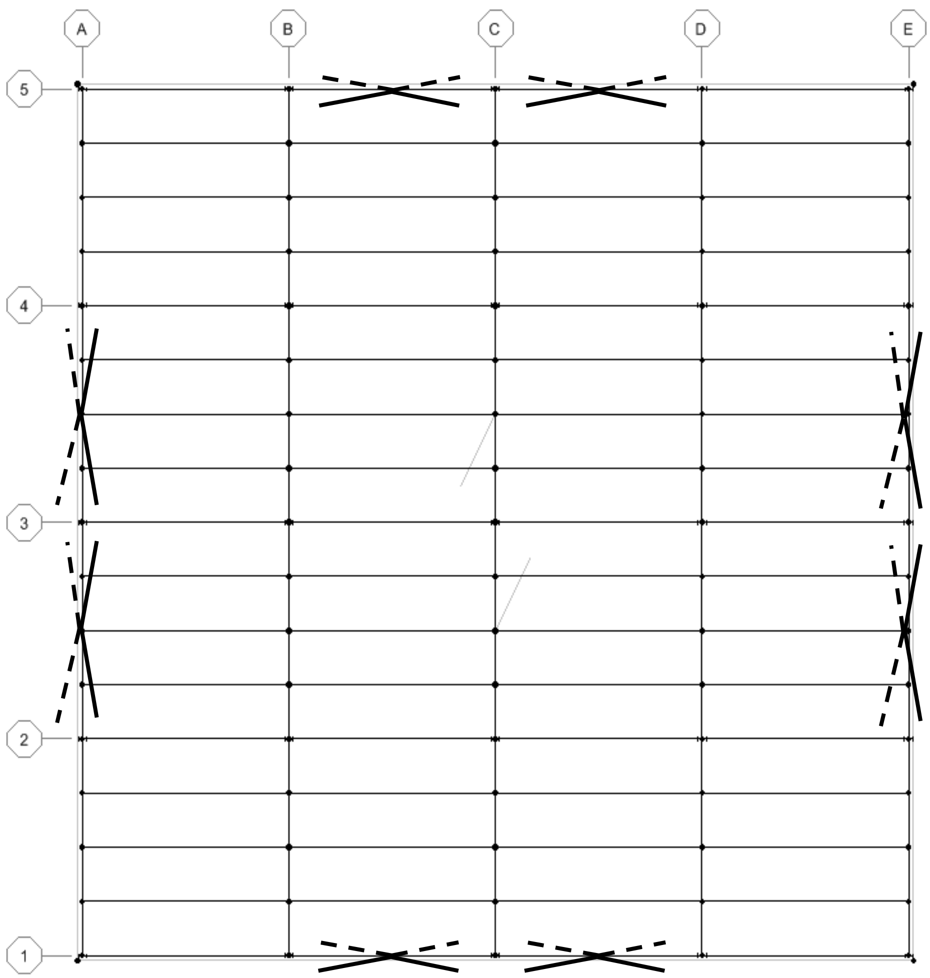

Figure 2: Floor Framing Layout

The vertical-load-carrying system consists of normal weight concrete fill on steel deck floors supported by steel beams and girders that span to steel columns. The bay spacing is 30 ft (9.1 m) each way, and there are three intermediate floor beams per bay. All beams and girders are composite.

The lateral-load-resisting system is comprised of special steel concentrically-braced frames (SCBF) for the United States code designs and moderately ductile concentrically braced frames (MD-CBF) for the Canadian code design. Round HSS sections are used for the brace elements in all designs. Beam-to-column connections throughout the building frame systems (including the braced bays) are considered pinned.

The foundation soil is representative of Site Class D conditions. For the American code designs, the building is located in Bellingham, WA, where the design roof snow load is 20 psf (1.0 kPa) per ASCE 7-05. To minimize the potential for differential ground motion hazards, the corresponding Canadian building is sited just across the border in Langley, British Columbia. It is designed according to the applicable Canadian building codes. In that location the design roof snow load is a slightly higher 34.2 psf (1.6 kPa) per NBCC 2005.

The building framing layout is very regular; the structure does not possess any horizontal or vertical irregularities. Because there are two braced bays at each line of perimeter framing the redundancy factor (ρ) is taken as 1.0 per ASCE 7-05 Sec. 12.3.4.2b.

III. Loads and Materials

Roof live load L_r	= 20 psf (1.0 kPa)
Roof dead load D	= 90 psf (4.3 kPa)
Exterior wall cladding	= 10 psf (0.5 kPa)
Floor live load L	= 50 psf (2.4 kPa)
Partitions	= 10 psf (0.5 kPa)
Floor D	= 87 psf (4.2 kPa)
Floor L reductions per the IBC or NBCC	

Roof dead load includes roofing, insulation, normal weight concrete-filled metal deck, concrete ponding allowance, framing, mechanical and electrical equipment, ceiling, and fireproofing. Floor dead load includes lightweight concrete-filled metal deck, ponding allowance, framing, mechanical and electrical equipment, ceiling, and fireproofing. Partition loads are considered live loads per ASCE 7-05 Sec. 4.2.2 (due to potential for rearrangement) but are treated as dead loads per NBCC 2005 Sec. 4.1.4.1.1. NBCC 2005 Sec. 4.1.4.1.5 requires that the partition load not be considered in load combinations used to determine overturning, uplift, sliding, failure due to stress reversal, and anchorage requirements. Regardless, the seismic weight of a typical tower floor, whose footprint is a square 14,722 ft^2 (1,368 m^2) in area, is 87 + 10 + 10(12)(4)(4)(30)/14722 = 100 psf (4.8 kPa).

Materials in all three designs are assumed to conform with the following specifications:

Concrete for floors	f'_c = 4 ksi (C28), normal weight (NW)
Structural steel	
Wide flange sections	ASTM A992, Grade 50
Round HSS sections	ASTM A500, Grade B

IV. Method Description

To compare the building code design requirements, three separate designs were performed:

- Design A: US building-code design using 2005 standards.
- Design B: US building-code design using 2010 standards.
- Design C: Canadian building-code design using 2005 standards.

In all cases seismic, rather than wind forces govern the building's lateral design (in part due to the mass of the thick concrete-filled decks). An equivalent lateral force (ELF) analysis (also known as equivalent static force analysis, or ESF, in Canada) is performed first to ensure proper scaling of the base shear in the subsequent modal response spectrum analysis (MRSA) used for strength design of the braces.

Design parameters used to establish the base shear are summarized in Table 1.

Table 1: Seismic Design Parameters

	Codes	R_d	R_o	R	Ω_o	C_d	T_{design}	$V_{ELF/ESF}$	V_{MRSA}
Design A	ANSI/AISC 341-10 SEI/ASCE 7-10			6.0	2.0	5.0	0.50	0.120W	0.102W
Design B	ANSI/AISC 341-10 SEI/ASCE 7-10			6.0	2.0	5.0	0.53	0.119W	0.101W
Design C	CAN/CSA S16S1-05 NRCC/CNRC NBCC 2005	3	1.3	3 x 1.3 = 3.9			0.44	0.199W	0.198W

Where

R_d is the Ductility-Related Force Modification Factor

R_o is the Overstrength-Related Force Modification Factor

R is the Response Modification Coefficient

Ω_o is the System Overstrength Factor

C_d is the Deflection Amplification Factor

T_{design} is the design fundamental period of vibration (as determined by code or structural analysis)

$V_{ELF/ESF}$ is the design seismic base shear as determined by the Equivalent Lateral Force (ELF) Procedure or the Equivalent Static Force (ESF) Procedure

V_{MRSA} is the design seismic base shear used in the Modal Response Spectrum Analysis (MRSA)

Design A: AISC 2005.

The applicable codes for this design are ASCE 7-05 (ASCE, 2005) and AISC 341-05 (AISC, 2005). The former covers the derivation of basic strength design requirements and drift limits, while the latter addressed detailing and proportioning issues. AISC 341 also requires higher strength for certain elements.

The building is located in a seismically active area. The Maximum Credible Earthquake accelerations are:

$S_{MS} = 1.083$ at 0.2 seconds, and

$S_{M1} = 0.570$ at 1.0 seconds.

Brace Design:

Braces are designed to resist the forces corresponding to the design base shear. The governing load combination is:

$$(1.2+0.2S_{DS})D + \rho Q_E + 0.5L + 0.2S$$

Where

S_{DS} is the design short-period acceleration

ρ is a "redundancy" factor (1.0 in this design)

D is Dead Load

L is Live Load

S is Snow Load

Beam and Column Design:

Beams and columns are designed based on the amplified seismic load. For both, the governing load combination is:

$$(1.2+0.2S_{DS})D + \Omega_o Q_E + 0.5L + 0.2S$$

See Figure 3 for the elevation of a typical frame. Note that the beams are considered laterally (and torsionally) braced at their third points.

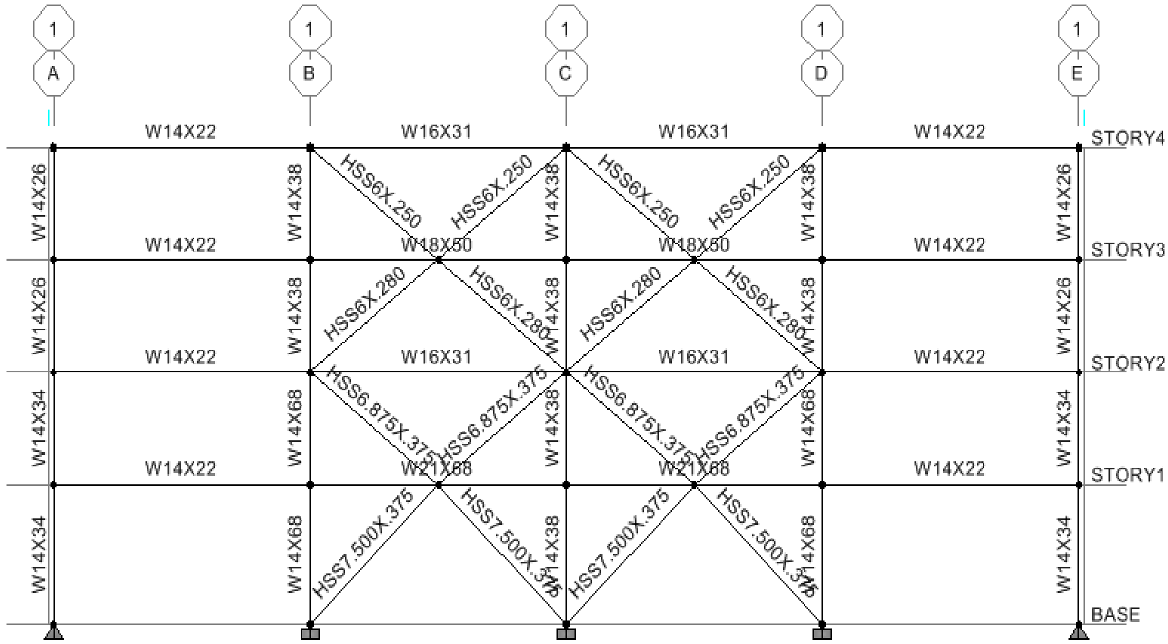

Figure 3: Elevation of Typical AISC 2005 Braced Frame

Design B: AISC 2010.

The applicable codes for this design are ASCE 7-10 (ASCE, 2010) and AISC 341-10 (AISC, 2010). At the time of writing these are both draft documents that update the 2005 editions of the standards.

The building is located in a seismically active area. The Risk-Targeted Maximum Credible Earthquake accelerations are:

S_{MS} = 1.071 at 0.2 seconds, and

S_{M1} = 0.614 at 1.0 seconds.

Brace Design:

Braces are designed to resist the forces corresponding to the design base shear. The governing load combination is:

$$(1.2+0.2S_{DS})D + \rho Q_E + 0.5L + 0.2S$$

Beam and Column Design:

AISC 341-10 requires beams and columns to be designed for the maximum forces that the braces can deliver. This requires the engineer to perform a plastic mechanism analysis. Maximum tension forces are defined as $R_yF_yA_g$ and maximum compression forces as $1.14P_n$ [determined using $RyFy$]. See Figure 4.

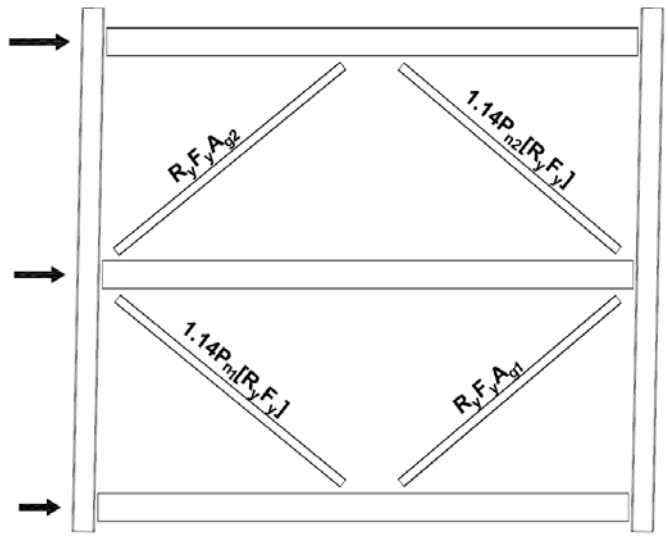

Figure 4. AISC 341-10 Maximum Brace Forces Mechanism

Additionally, because of the dramatic force redistributions resulting from brace buckling, AISC 341-10 requires a second plastic mechanism analysis in which braces in tension achieve their expected tension strength and braces in compression have lost most of their capacity. Maximum tension forces are again defined as $R_yF_yA_g$ and post-buckling compression forces as $0.3P_n$. See Figure 5.

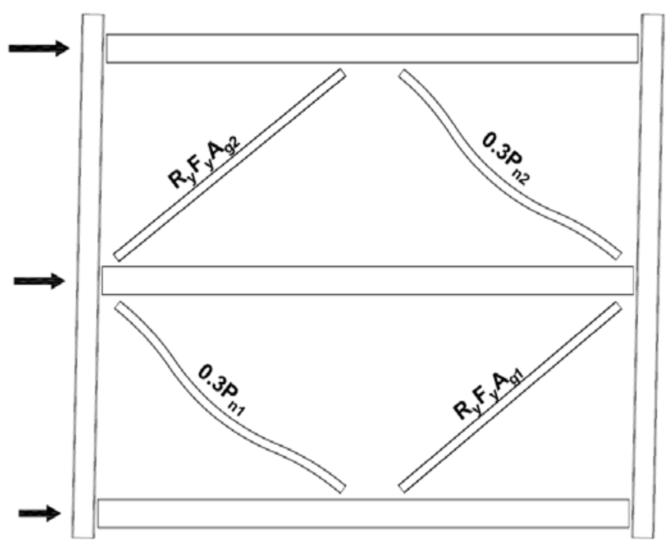

Figure 5. AISC 341-10 Post-Buckled Brace Forces Mechanism

Note that the redistributions cause large axial force in the beams at the third floor and the column at the center of the two-bay frame.. Note also that the elastic-analysis-based 2005 method determined essentially zero seismic force for these members (aside from the small vertical seismic load effect).

Beams, as collectors, must also be designed for the amplified seismic load.

See Figure 6 for the elevation of a typical frame.

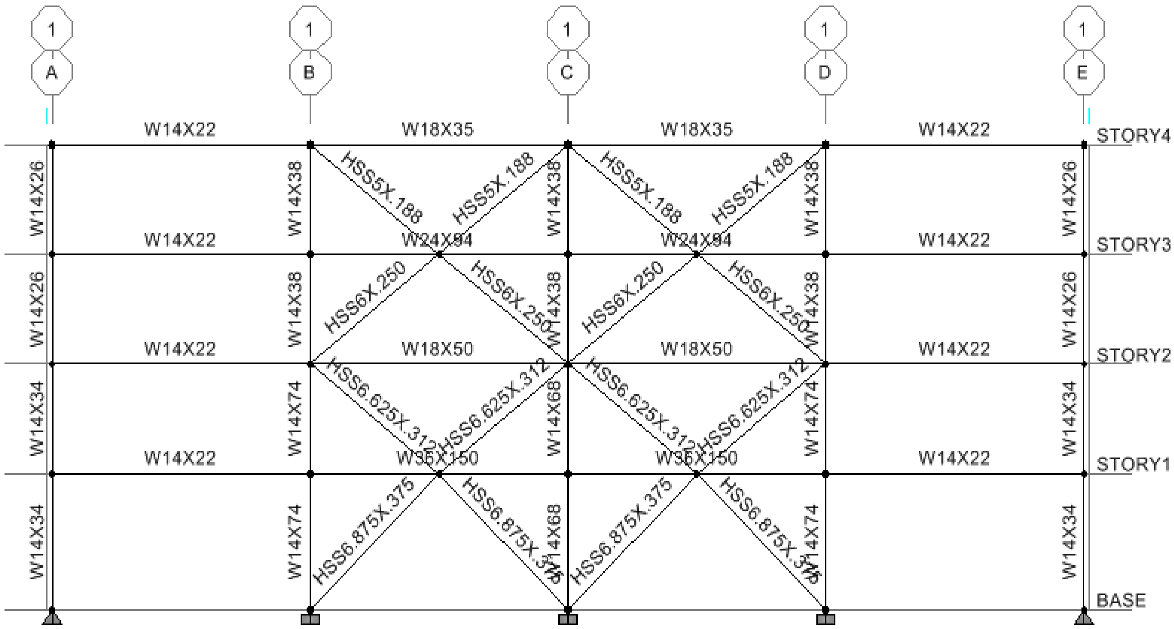

Figure 6: Elevation of Typical AISC 2010 Braced Frame

Design C: CSA.

The applicable codes for this design are NBCC/CSA 2005 (NRCC/CNRC, 2005) and CAN/CSA S16S1-05 (CSA, 2005). These documents play the same respective roles as do the ASCE 7 and AISC documents in the US.

The NBCC/CSA 2005 Canadian provisions do not allow for nearly as much inelastic action as the AISC 2005 and 2010 provisions do, and require the braced frames to resist a greater share of the design seismic displacement elastically.

The design spectrum corresponding to the Maximum credible Earthquake is defined by the following:

S(0.2 sec) = 1.166

S(0.5 sec) = 0.831

S(1.0 sec) = 0.386

S(2.0 sec) = 0.199

S(4.0 sec) = 0.100

Brace Design:

Braces are designed to resist the forces corresponding to the design base shear. The governing load combination is:

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

Beam and column design:

The beams and columns within the braced frames are designed in a manner similar to Design B, except that in determining E the maximum capacity of the braces is defined somewhat differently. In tension, the brace capacity is set equal to $R_yF_yA_g$, except a value of $0.6R_yF_yA_g$ is permitted to be used in buildings of four or fewer stories. In compression, the maximum force is either $1.2P_n$ [determined using $RyFy$] (just prior to onset of buckling) or 0.2 $R_yF_yA_g$ (post-buckled) in compression per CSA-S16-01 Sec. 27.5.2.4. See Figures 7 and 8 for the mechanisms.

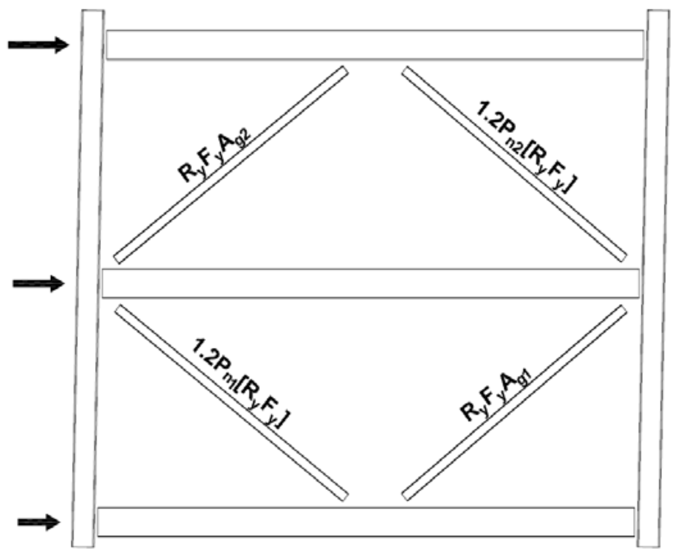

Figure 7. NBCC 2005 Maximum Brace Forces Mechanism

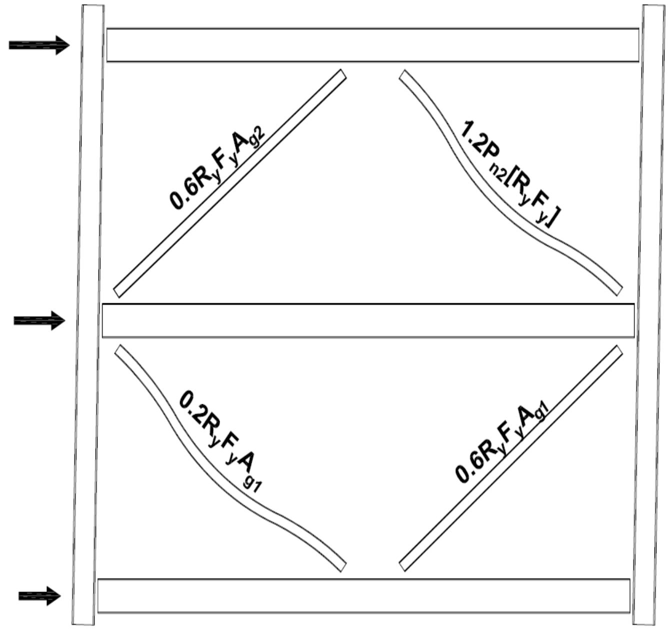

Figure 8. NBCC 2005 Post-Buckled Brace Forces Mechanism

See Figure 9 for the elevation of a typical frame.

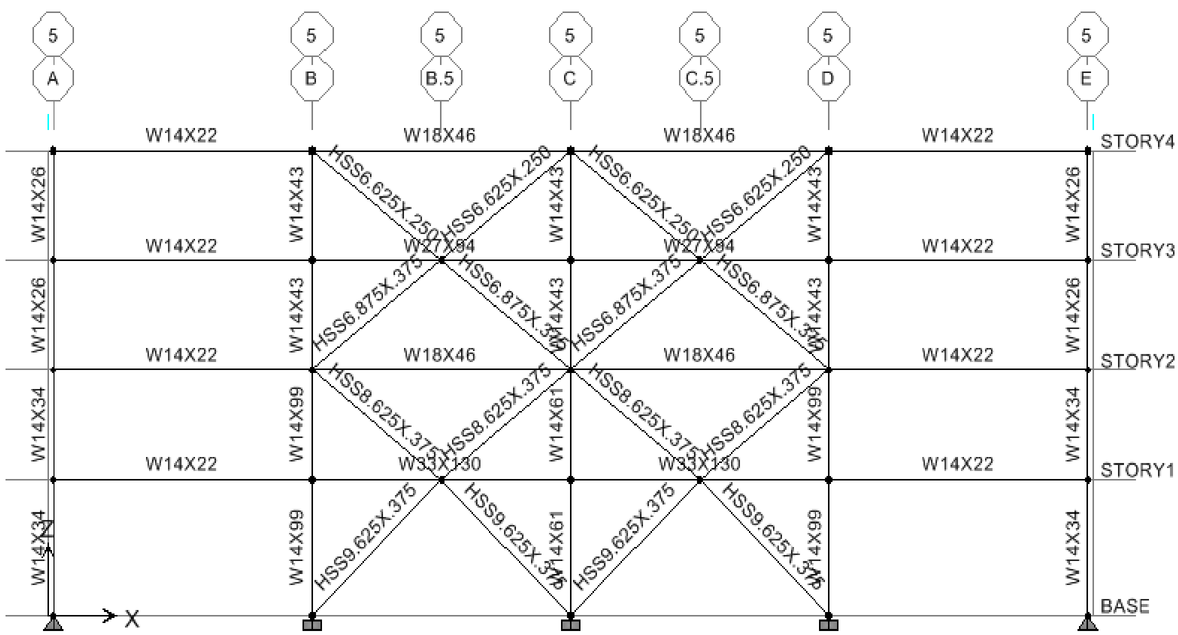

Figure 9: Elevation of Typical NBCC 2005 Braced Frame

V. Example Designs

Figure 3 shows a typical braced frame elevation designed per the AISC 2005 provisions. Braces are sized by response spectrum analysis to meet 85% percent of the static base shear demand of 0.120W and must meet compactness requirements of AISC 341-05 Table I-8-1. Note that the size of the braces in the uppermost story are controlled by the AISC 341-05 Sec. 13.2a requirement that $Kl/r \leq 4 \times \mathrm{SQRT}(E/F_y)$. Columns in the braced frame bays are designed for load combinations including the amplified seismic load effect with $\Omega_o = 2.0$. They too must be seismically compact per AISC 341-05 Table I-8-1. W14x38 sections are sufficient throughout the braced frames except at the outermost columns in the lower two stories, where W14x68 sections are necessary to meet the demands. Similarly, beams are designed for the amplified seismic load effect in conjunction with an additional force where the braces intersect a beam at midspan as required by AISC 341-05 Sec. 13.4a. As such, very light beam sections (W16x31) are found to be adequate at the two levels where braces frame directly into the columns and do not frame into beams. Recall that beams are considered to be laterally (and torsionally) supported at their third points for all three frames discussed herein. Figure 6 shows a typical braced frame elevation designed per the AISC 2010 provisions. As before, braces are sized by response spectrum analysis to meet 85% of the static base shear demand of 0.119W. However, the different brace sizes (relative to the AISC 2005 design) result from both the slightly lower base shear together with the heavier (and stiffer) column sections required by the AISC 2010 provisions. Column sizes are determined considering the maximum forces that the braces can deliver in tension and compression. Post-buckled brace behavior is examined as well, requiring the middle columns adjoined by braces on both sides to be designed for much larger loads than required by AISC 2005. Accordingly, larger W14x74 and W14x68 are necessary to meet maximum expected force demands at the outermost and middle columns, respectively, in the lower two stories. The same compactness limitations that governed the design of braces and columns in the AISC 2005 frames apply to the AISC 2010 frames. Like the columns, beams within the braced frame bays are also designed considering the force transfer mechanisms associated with the braces reaching their maximum tensile strengths together with either their maximum or post-buckled compression strengths, depending on which controls. For this reason, heavier beam sections are required at all levels relative to the AISC 2005 design.

Finally, Figure 9 shows a typical braced frame elevation designed per the NBCC 2005 provisions. Braces are sized by response spectrum analysis to meet 80% of the significantly larger static base shear demand of 0.199W. CSA-S16-01 requires braces to have $Kl/r \leq 200$ and $b/t \leq 10{,}000/F_y$ or that required of Class 1 sections. Like the columns in the AISC 2010 braced frames, columns in the NBCC 2005 braced frames are designed to accommodate maximum and post-buckled capacities of the braces that are slightly different (as outlined in CSA-S16-01 Sec. 27.5.4.2). Columns within braced frames must be Class 1 or Class 2 sections per CSA-S16-01 Sec. 27.5.5.2. The beams are also designed using a mechanism approach with brace capacities dictated by CSA-S16-01 Sec. 27.5.2.4 and considering a minimum diaphragm force equal to the base shear divided by the number of stories per NBCC 2005 Sec. 4.1.8.15.(1b). Beams must be Class 1 sections to use the reduced $0.6R_yF_yA_g$ brace tensile strengths permitted by CSA-S16-01 Sec. 27.5.2.4. Generally speaking, the NBCC 2005 braced frame consists of substantially heavier sections to meet higher elastic base shear demands relative to the AISC 2005 and 2010 designs.

VI. Comparison

The Canadian standards result (Design C) in a substantially heavier design with lower inelastic demand. Between the two US designs, the 2010 mechanism-based design has slightly higher tonnage. Also, the braces represent a slightly lower proportion of that tonnage in the 2010 design; the increase is in the beam and column weights.

To study the effectiveness of these code provisions, simplified push-over analyses were performed of each of these frames using a first-mode force distribution. A simple brace-element model based on ASCE 41 (ASCE, 2006) recommendations was used to represent brace nonlinear behavior in tension and compression. No other nonlinear behavior was simulated.

Design A reached a roof drift of 1.25 in (3.18 cm) [0.2%] before the shared middle column buckled in compression at the first story due to the accumulation of unbalanced forces. Prior to the onset of this catastrophic failure, the compression braces at the third and first stories had buckled. The axial forces in the beams at the third floor reached 135 kips (600 kN), more than double the force they were designed for, prior to the onset of buckling at the first story column. In the shared column they reached 232 kips (1,032 kN), the buckling capacity of the first story segment. The maximum base shear achieved was 935 kips (4,160 kN). Figure 10 shows the maximum forces in each element as a percentage of the design forces. In this and each of the following maximum frame force figures, "NL(C)" indicates a brace has buckled in compression while "NL(T)" represents stable tensile yielding in a brace.

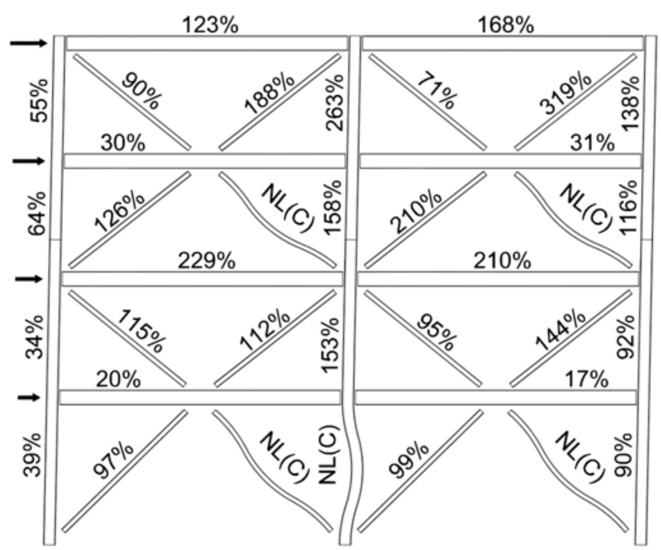

Figure 10: Maximum Forces in AISC 2005 Braced Frame Relative to Design Forces

Design B achieved the design mechanism (braces yielding or buckling, with no axial ductility in beams or columns). The maximum base shear achieved was 1,286 kips (5,720 kN) at a roof drift of 3.34 in (8.50 cm) [0.5%], when the analysis indicated compressive brace failures at the fourth, third, and first stories. Figure 11 shows the maximum forces in each element as a percentage of the design forces. The methodology appears to have correctly determined maximum forces for many of the elements underdesigned in Design A. In particular, the column shared by the two braced bays and the beams at the third floor. However, the methodology also increased the over design of the columns at the ends of the braced frame.

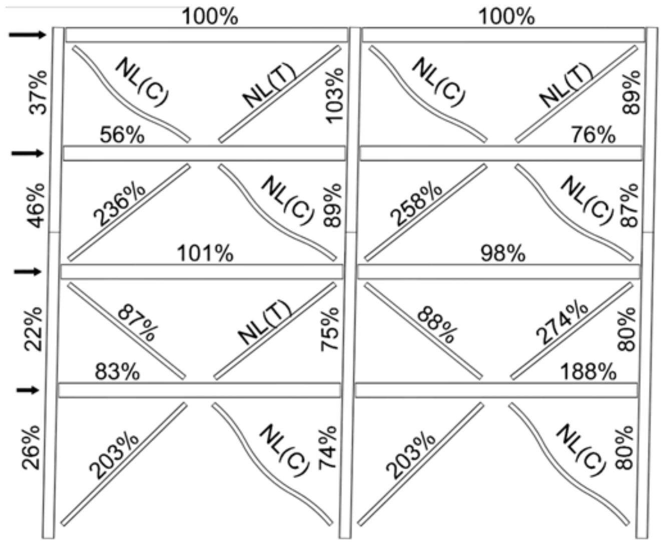

Figure 11: Maximum Forces in AISC 2010 Braced Frame Relative to Design Forces

Design C also appeared to achieve the design mechanism. The maximum base shear achieved was 2,500 kips (11,120 kN) at a roof drift of 6.75 in (17.15 cm) [1.1%], when the analysis indicated compressive brace failures at the third and first stories. Figure 12 shows the maximum forces in each element as a percentage of the design forces. Column forces appear to be adequately addressed by this methodology. However, it appears that beam collector forces and beam transfer forces resulting from post-elastic redistribution of forces after brace buckling significantly exceed the design forces.

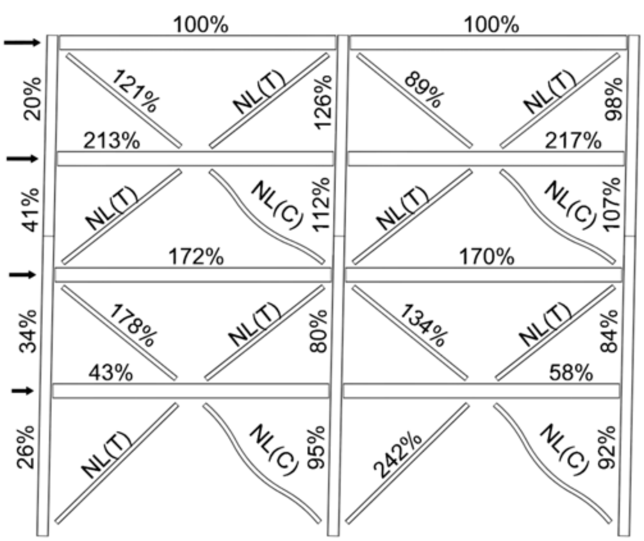

Figure 11: Maximum Forces in NBCC 2005 Braced Frame Relative to Design Forces.

It appears that the changes in the AISC 341 documents are at least partially justified by the change in the overall limiting mode of the frame. Whether the new rules might justify a reduction in design force was not investigated.

The Canadian design approach differs substantially from either of the U.S. designs in the degree of ductility that is allowed. This approach appears to be more reliable in terms of performance, but at a significant premium in terms of steel tonnage.

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