11th Pacific Structural Steel Conference
Shanghai, China, October 26-28, 2016

COMPARISON OF SEISMIC DESIGN REQUIREMENTS FOR STEEL MOMENT RESISTING FRAMES WITH EMPHASIS ON STABILITY OF COLUMNS IN NORTH AMERICA, NEW ZEALAND, AND EUROPE

Ali Imanpour*, Dimitrios Lignos, Charles Clifton***, and Robert Tremblay******

* Department of Civil Engineering & Applied Mechanics, McGill University, Montréal, Canada
e-mail: ali.imanpour@mail.mcgill.ca

** School of Architecture, Civil & Environmental Engineering, Swiss Federal Institute of Technology, Lausanne (EPFL), Switzerland
e-mail: dimitrios.lignos@epfl.ch

*** Department of Civil and Environmental Engineering, University of Auckland, Auckland, New Zealand
e-mail: c.clifton@auckland.ac.nz

****Department of Civil, Geological & Mining Engineering, Polytechnique Montréal, Canada
e-mail: robert.tremblay@polymtl.ca

Keywords: Columns, Steel moment frames, Seismic design, Stability.

Abstract. *The seismic design provisions and stability requirements for steel columns used at the first storey of MRFs as specified is Canadian steel design standard (CSA S16), U.S. design provisions for steel buildings (AISC 341 and AISC 360), the New Zealand steel structures standard (NZS 3404), and Eurocodes 3 and 8 (EC3 and EC8) are reviewed. The code provisions are applied to a deep, slender steel I-shaped column to identify possible failure modes. A finite element analysis of the column is performed under cyclic inelastic lateral drift history to validate code predictions. The column was found to fail by out-of-plane instability, after local buckling has developed in the base plastic hinge. This failure mode was not expected from the axial-bending interaction equations available in the design standards except the equation of Eurocode 3. Limits for out-of-plane slenderness in Canadian and European provisions indicate the observed failure mode.*

1 INTRODUCTION

Steel moment resisting frames (MRFs) have been popular in seismic regions due to their large ductility capacity and their architectural flexibility. Several research studies have been performed to determine the inelastic cyclic response of steel beam-to-column moment connections and columns [1 - 4], which led to development of seismic design provisions for steel MRFs and adaptation in steel design standards around the world. In steel MRFs with fully restrained connections, beam flexural yielding is preferred over column plastic hinging because columns carry axial compression loads in addition to flexural bending demands and the strong column forces inelastic demand throughout the frame rather than concentrating it at a storey level. As such, current design provisions require that columns in steel MRFs be designed such that they can resist combined axial force and lateral drift demands due to seismic effects; also, plastic hinging is permitted at the base of the columns in any multi-storey building and the columns should be designed and detailed so that they will not become unstable if plastic hinging occurred at their bases during the earthquake. Steel design standards worldwide have adopted various approaches to verify strength and stability of MRF columns in order to ensure satisfactory performance under gravity and seismic load effects. In the last decades, deeper and slender wide-flange columns have become common for MRFs, essentially to more effectively control building drifts. Based

on recent analytical and experimental findings [5 - 8], dominant limit states observed in steel columns may not be appropriately addressed using the design approaches adopted in current design guidelines. The limit states include out-of-plane instability near plastic hinge regions and out-of-plane buckling along the height combined with torsional deformations. This paper reviews the seismic design provisions for steel columns that are specified in Canada, the U.S., New Zealand, and Europe. In particular, strength and stability requirements specified for deep I-shaped columns at the first level of MRF buildings designed for ductile seismic response are described. These provisions are then applied to and compared for a I-shaped column part of a MRF to highlight differences between the various design approaches specified. The dominant limit state(s) are also identified for each standard. Finally, the results of finite element analysis (FEA) are presented for the same column under constant axial compression and cyclic lateral drift demand. The observed failure modes from FEA are compared to the limit states predicted by the design codes.

2 SEISMIC DESIGN PROVISIONS AND STABILITY DESIGN CRITERIA

Columns in moment resisting frames are subjected to combined axial compression and bending in the plane of the frame. The columns are generally wide flange rolled sections that are fixed at their bases and oriented such that bending occurs about strong-axis. As shown in Figure 1a, ductile MRFs are designed to dissipate seismic input energy during severe earthquakes by means of plastic hinges forming at the beam ends and at the column bases. Beams are sized to resist end moments due to gravity loads plus code specified seismic loads as obtained from elastic analysis. Columns at the first storey are also verified for that loading condition, especially to ensure that base yielding does not occur under code seismic loads (Figure 1b). Additionally, they must be designed so that they remain essentially elastic above the base plastic hinges (Figure 1c) to avoid a complete plastic lateral mechanism at the first level, also referred to as soft-storey response. This section presents a review of the strength and stability requirements for steel columns used at the first level of multi-storey moment frames that are specified in CSA S16-14 [9], AISC 341-10 and 360-10 [10, 11], NZS 3404 [12], and EC3 and EC8 [13, 14]. In modern seismic design guidelines for steel structures, first-storey columns in steel MRFs are verified to: 1) meet cross-section slenderness limits in order to delay/minimize local buckling; 2) satisfy drift limits within the storey; 3) have sufficient strength to resist the code specified seismic and gravity loads (Figure 1b); 4) satisfy Strong-Column/Weak-Beam (SCWB) criterion at the storey joint to promote plastic hinges in the beams; and 5) remain stable in and out of the plane of the frame after formation of plastic hinges in beams and at their bases (Figure 1c). The latter is not explicitly required in all seismic design standards. In some standards, additional design requirements are specified to achieve stable column response after the expected plastic hinges have developed. This section focuses on the SCWB criterion and member stability requirements found in the aforesaid design standards.

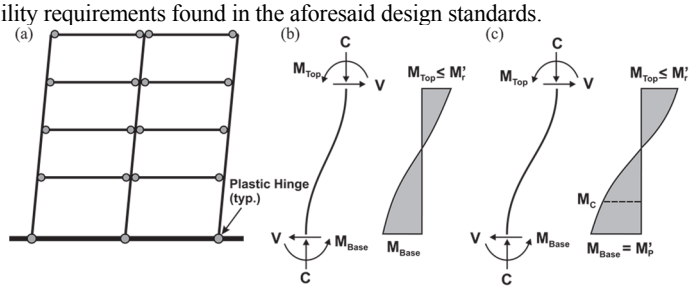

Figure 1. (a) Plastic mechanism in a MRF; (b) and (c) first-storey column moment diagram under specified seismic load and when a plastic mechanism has formed, respectively.

2.1 CSA S16-14

CSA S16-14 contains seismic design requirements for various MRF categories. The provisions for the most ductile category (Type D) are discussed herein. For these frames, the SCWB criterion at every joint is

verified by comparing the sum of the column flexural resistances below and above the joint to the sum of the moments imposed by the beams reaching their probable flexural resistances:

$$\sum 1.18\,\phi M_{\mathrm{pc}}\left(1-\frac{C_f}{\phi C_{\mathrm{y}}}\right)\geq\sum\left(1.1R_{\mathrm{y}}M_{\mathrm{pb}}+V_{\mathrm{h}}s_{\mathrm{h}}\right) \tag{1}$$

where ϕ is the resistance factor for steel ($\phi = 0.9$), M_{pc} and M_{pb} are the plastic moment resistances of the columns and beams about their strong-axes, respectively, C_{f} is the factored axial compression in the columns, and C_{y} is the column axial yield strength (equal to AF_{y}, where A is the column cross-sectional area and F_{y} is the steel yield strength). The term $1.1R_{\mathrm{y}}M_{\mathrm{pb}}$ represents the probable, or expected, moment resistance of the beams, including strain hardening effect (factor 1.1) and the difference between probable and nominal steel yield strengths (factor R_{y}). The total flexural demand on the columns also includes the moment induced by beam shear forces V_{h} acting in beam plastic hinges located at distance s_{h} from the column centreline. For the first-storey columns where base plastic hinging is expected, columns must meet the requirements for Class 1 (plastic design) sections. They must also be laterally braced using:

$$\frac{L_{\mathrm{cr}}}{r_{\mathrm{y}}}=\frac{17250+15500\,\kappa}{F_{\mathrm{y}}} \tag{2}$$

where L_{cr} is the column unsupported length, typically the storey height, r_{y} is radius of gyration about weak-axis, and κ is the ratio of the smaller to the larger factored column end moments (positive if double curvature). When verifying Eq. 2, κ must be set equal to zero representing the intermediate, more critical condition during an earthquake where the moment is maximum at one end and near to zero at the opposite end. It is noted that Eq. 2 is similar although more stringent than the equation for $L_{\mathrm{cr}}/r_{\mathrm{y}}$ used for plastic design in CSA S16. Furthermore, in moderate and high seismic regions, the column axial load must not exceed 0.30 C_{y} to maintain the column flexural ductility.

CSA S16 does not contain member stability requirements other than Eq. 2. The S16 interaction equation for columns part of elastic frames could be used with modifications for this purpose. For members subjected to factored axial compression C_{f} and factored strong-axis bending moment M_{fx}, this equation is:

$$\frac{C_{\mathrm{f}}}{C_{\mathrm{r}}}+\frac{0.85\,U_{1\mathrm{x}}\,M_{\mathrm{fx}}}{M_{\mathrm{rx}}}\leq 1.0,\ \ where\ \ U_{1\mathrm{x}}=\frac{\omega_{1\mathrm{x}}}{1-C_{\mathrm{f}}/C_{\mathrm{ex}}} \tag{3}$$

where C_{r} and M_{r} are respectively the column factored axial and moment resistances, $\omega_{1\mathrm{x}}$ is the bending coefficient, and C_{ex} is the column Euler (elastic) buckling load. The factor $U_{1\mathrm{x}}$ therefore accounts for moment gradient and member level second-order effects. By selecting appropriate values for these parameters, the equation is used to verify the following three limit states: yielding of cross-section, in-plane buckling, and lateral-torsional (out-of-plane) buckling. For moment frame columns, cross-section strength is already verified by the SCWB criterion and Eq. 3 would be used to verify the remaining two failure modes. For elastic unbraced frames, global stability relies on the column flexural stiffness and $U_{1\mathrm{x}}$ is set equal to 1.0 for both cases. For frames responding in the nonlinear range, frame stability essentially depends on the frame lateral yield strength and post-yield stiffness and $U_{1\mathrm{x}}$ can be determined as shown in Eq. 3. The force C_{f} is same as considered in the SCWB check and the moment M_{fx} corresponds to the maximum probable flexural strength at the plastic hinge location, including strain hardening and probable yield strength effects:

$$M_{\mathrm{Base}}=1.18\left(1.1R_{\mathrm{y}}M_{\mathrm{pc}}\right)\left(1-\frac{C_{\mathrm{f}}}{R_{\mathrm{y}}C_{\mathrm{y}}}\right) \tag{4}$$

For the condition of Figure 1c, the column is in double curvature, which gives $\omega_{1\mathrm{x}} = 0.4$. C_{r} is computed about strong- and weak-axes, respectively, for in-plane and out-of-plane buckling modes. For the former, M_{r} is based on flexural yielding while lateral-torsional buckling is considered for the second verification.

2.2 AISC 341-10 and AISC 360-10

The seismic requirements of AISC 341-10 for columns part of Special Moment Frames (SMFs) are described in this section. The column sections must meet the stringent limits for highly ductile members. The SCWB criterion is same as in CSA except that the resistance factor and the factor 1.18 are omitted. AISC 341 does not contain any special requirements for columns that must develop flexural plastic hinges at their bases. In particular, column out-of-plane stability is not explicitly required to be verified, which may be attributed to the fact that MRF columns in the past were typically heavy and stocky I-shaped members.

For deeper and more slender columns, verification of the stability could be performed using the interaction equation for members subjected to combined axial and bending of the AISC 360-10 (note that the CSA notation is used throughout the article to ease comparison):

$$\frac{C_f}{C_r}+\frac{8}{9}\frac{B_{1x}M_{fx}}{M_{rx}}\le 1.0 \quad when\,\frac{C_f}{C_r}\ge 0.2;\quad \& \quad \frac{C_f}{2C_r}+\frac{B_{1x}M_{fx}}{M_{rx}}\le 1.0 \quad when\,\frac{C_f}{C_r}<0.2 \tag{5}$$

In this equation, C_r is the compressive strength for weak-axis buckling, M_{rx} is the flexural strength accounting for lateral-torsional buckling, and B_{1x} is same as U_{1x}. As in CSA S16, B_{1x} must not be less than 1.0 for elastic design; for inelastic frame response, B_{1x} can take the value of U_{1x} (Eq. 3). The moment M_{fx} would also be the base moment (Eq. 4), except that the factor 1.18 would be omitted for consistency.

2.3 NZS 3404

The New Zealand steel structures standard (NZS 3404) provides detailed design provisions for MRF columns. Four structural categories are allowed in this standard for MRFs. The requirements for limited ductile (Category 2) moment frames are covered herein, as this system is more common than the fully ductile (Category 1) one. In NZS 3404, columns in MRFs must comply with the plate element slenderness limits within the yielding regions and correspond to Category 2 members. The SCWB criterion at joints in is same as in Eq. 1 except that the moments imposed by the beams at the joints is augmented by an overstrength factor ϕ_{oms}. For steels with a yield strength of 350 MPa, ϕ_{oms} = 1.3, which is larger than $1.1R_y$ = 1.21 considered in Canada. The contribution from the floor slabs must also be included unless the slab is separated from the columns.

NZ3404 contains several stability requirements for first-storey MRF columns that are expected to develop flexural yielding at their bases. The columns should have Full Lateral Restraint (FLR), i.e., they can develop their full plastic moment capacity without lateral-torsional buckling over the storey height. This is verified by $\alpha_m\alpha_s \ge 1.0$, where α_m and α_s are respectively the moment modification factor and the slenderness reduction factor for flexural resistance. In that verification, α_m must not exceed 1.75, which represents the case similar to the condition assumed when applying Eq. 2 in S16. NZS 3404 also requires that full lateral restraint be provided to both column flanges within the length of a yielding region, L_{yr}, at a maximum spacing L given by:

$$\frac{L}{r_y}=k_{yr}\sqrt{\frac{250}{F_y}} \tag{6}$$

where k_{yr} is the yielding region restraint spacing coefficient (= 30 and 40 for categories 1 and 2 I-shaped member). One restraint must be a physical restraint (e.g. at the base connection) while the specified end of the yielding region away from the column ends is considered a second point of restraint for this check. The yielding region for columns is the length over which the moment exceeds 0.75 to 0.85 times the column flexural capacity, depending on C_f, with a minimum of 1.5 times the column depth.

Finally, three column axial force limits are prescribed in the NZS 3404 for MRF columns. The first limit aims at achieving stable inelastic flexural response. For Category 2 members, it is equal to $0.7\phi C_y$, with ϕ = 0.9, which is higher than the limit of $0.3C_y$ specified for Type D MRFs in S16 (note that NZS limit reduces to $0.45C_y$ for Category 1 members, which is closer to the CSA S16 limit). The second limit on $C_f/\phi C_y$, known as the End Yielding Criteria (EYC), aims at preventing the moment at any point over the

column length from exceeding the base moment, i.e. $M_c < M_{base}$ in Figure 1c, which can be verified by satisfying:

$$\frac{C_f}{\phi C_y} \le \left[\frac{0.263(\kappa+1)^{0.88}}{e^{(0.19/(\kappa+1))}} \right]^{\lambda_{EYC}} \quad where: \ \lambda_{EYC} = \sqrt{\frac{C_y}{C_{ex}}} \tag{7}$$

where λ_{EYC} is the non-dimensional slenderness ratio of the column for strong-axis buckling and κ is the end moment ratio. When verifying Eq. 7, the ratio κ is taken equal to zero to represent the aforementioned moment gradient condition. The third limit on column axial load involves the axial force induced by gravity loading alone, C_{fg}, and aims at preventing local buckling of the web to shortening of the column [15]:

$$If \ d/w > 30\sqrt{F_y/250}: \quad \frac{C_{fg}}{\phi C_y} \le 1.91 - \left[\frac{d}{w} \frac{\sqrt{F_y/250}}{27.4} \right] \le 1.0 \tag{8}$$

In NZS, there is no need to check the out-of-plane stability of a column using interaction equations such as those discussed for Canada and the U.S. because the strength of the section at the member ends should be critical when the three axial load limit checks are satisfied [16].

2.4 Eurocodes 3 and Eurocodes 8

Design requirements in EC8 [14] for ductility class DCH MRFs are described herein. Columns in which plastic hinging is expected must have class 1 cross-sections. As in previous codes, the columns must satisfy the SCWB criterion and resist the combined axial force and bending moments due to plastic hinging of the adjacent beams. This criterion is similar to that given by Eq. 1 except that the moment demand from the beams is equal to moments from gravity loads plus $1.1\gamma_{ov}M_{pb}$, where γ_{ov} is the material overstrength factor (= 1.25). The flexural resistance of the column cross-section under combined axial force and bending moment about strong-axis, M_r, is obtained from EC3 [13]:

$$M_{rx} = M_{pc}\left[\frac{1-n}{1-0.5a} \right] \le M_{pc} \tag{9}$$

where $n = C_f/C_y$ and a is the web to cross-section area ratio (but $a \le 0.5$). Note that M_r may be taken equal to M_{pc} if C_f is less than the minimum of $0.25C_y$ and half the force corresponding to the yielding of the web.

Unlike the other standards discussed here, EC8 explicitly requires that stability of the column be verified under the force demand corresponding to Figure 1c. The stability checks are performed based on EC 3 for in-plane and out-of-plane buckling limit states as follows:

$$\frac{C_f}{\chi_x C_y} + \frac{k_{xx}M_{fx}}{\chi_{LT}M_{pc}} \le 1.0 \ \ \& \ \ \frac{C_f}{\chi_y C_y} + \frac{k_{yx}M_{fx}}{\chi_{LT}M_{pc}} \le 1.0 \tag{10}$$

where M_{fx} is the base moment taken equal to M_{pc}, without reduction for axial loads, χ_x, χ_y, and χ_{LT} are the reduction factors for flexural buckling about strong-axis, flexural buckling about weak-axis, and lateral-torsional buckling, respectively; k_{xx} and k_{yx} are the interaction factors to account for member second-order effects, similar to factors U_{1x} and B_{1x} in Canada and U.S. provisions, respectively. As discussed for these two countries, values of k_{xx} and k_{yx} for seismic design can be determined using equivalent uniform moment factors from the actual ratio between the column end moments. For plastic design, EC3 includes stability requirements for members with plastic hinges that could be considered for first-storey MRF columns: lateral restraint must be provided at both flanges at the hinge location and the member length must not exceed the stable length. For members subjected to low axial load and a moment gradient corresponding to double curvature with EC3 end moment ratio, ψ, close to -1.0, the stable length is: $L_{Stable} = (60 - 40\psi) r_y \sqrt{F_y/250}$.

3 APPLICATION TO A DEEP AND SLENDER COLUMN

A W610x153 column (Imperial designation = W24x103) conforming to ASTM A992 with F_y = 345 MPa and having a height L = 4 m is selected to compare the design provisions described in Section 2. The column represents an interior first-storey column of a mid-rise steel MRF located in a high seismic region.

3.1 Design forces

Although different force demands are specified on the columns in the codes studied, it is assumed for this example that the column carries a factored axial compression load $C_f = 0.2C_y$ and that the moment demand from the beams is equal to $M_{fx} = 0.8M_{pc}$ when the full frame lateral plastic mechanism of Figure. 1c has formed. Because this is an interior column, the axial compression is essentially due to gravity loads. The base moment values are taken as described for the various codes.

3.2 Verification of the limit states

The strength and stability of the W610x153 column was checked in accordance with [9-14] and key results are presented and compared in Table 1. The effective column length factor was set equal to 1.0 to verify the stability of the member. In the table, an empty cell indicates that the design standard does not contain any requirement for the limit state being verified. Values in brackets represent the limits that should be satisfied for design standard.

For CSA S16, the cross-section complies with Class 1 limit. The SCWB ratio exceeds 1.0 which suggests that plastic hinging will be limited to the beams. The base moment is equal to 1.19 M_{pc}. As shown, the column has sufficient resistance to prevent in-plane and out-of-plane buckling when moment varies from that value at the base to 0.8 M_{pc} at the top end (Figure 1c). For these two buckling limit states, Eq. 3 would reach 1.35 and 1.51, respectively, if U_{1x} = 1.0 was used, as specified in S16 for elastic frames. It is noted that the expected yield strength (R_yF_y) and ϕ = 1.0 could be used when determining M_r in Eq. 3 to verify the in-plane and lateral-torsional limit states, as yielding is expected at the base of the column. The column global slenderness ratio limit (L_{cr}/r_y) is not satisfied for this column, which can be an indication of likelihood of column lateral instability about weak-axis. The axial force demand however satisfies the $0.3C_y$ limit.

Table 1. Design calculations for the W610x153 column.

Limit State	CSA S16	AISC 341 & AISC 360	NZS 3404	EC3 & EC8
Local Slenderness	Class 1	Highly Ductile	Category 2	Class 1
SCWB ratio	1.03 (> 1.0)	1.0 (> 1.0)	1.03 (> 1.0)	1.25 (> 1.0)
M_{Base}/M_{pc}	1.19	1.0	0.92	1.0
In-plane Stability	0.68 (< 1.0)	-	-	0.60 (< 1.0)
Out-of-plane Stability	0.84 (< 1.0)	0.56 (< 1.0)	-	1.11 (< 1.0)
Lateral Bracing	79 (< 50)	-	FLR is required	3834 mm (> 4000)
$\alpha_m\alpha_s$	-	-	1.33 (> 1.0)	-
L within L_{yr}	-	-	1720 mm (> 933)	-
Axial Force	$0.2C_y$(< $0.3C_y$)	-	$0.22C_y$ (< $0.7\phi C_y$)	-
Axial Force due to Gravity	-	-	$0.22C_y$ (< $0.25\phi C_y$)	
End Yielding Criteria	-	-	0.22 (< 0.73)	

When verifying the column using AISC 341, local slenderness ratios meet the limits for highly ductile members and the result is obtained for the SCWB check is similar to that obtained for S16. The base moment is lesser than in S16 but the column satisfies Eq. 5 for out-of-plane buckling. According to NZS 3404, the section slenderness does not exceed the limit specified for Category 2 members and the SCWB ratio is comparable to the corresponding values obtained with CSA S16 and AISC. Based on NZS 3404, FLR is satisfied for this column with $\alpha_m\alpha_s$ = 1.33. The length of the yielding region at the column base is 950 mm (> 1.5d) and the maximum spacing between restraints in that length is 1720 mm from Eq. 6. Hence, the restraint offered by the column base connection is sufficient to ensure stability of the yielding region.

Had the column been a Category 1 designed for large plastic rotation demands, the length L would have reduced to 1290 mm and still no additional restraint would have been needed. For this column, the NZS criteria for out-of-plane stability appear to be more liberal compared to the S16 lateral bracing (L_{cr}/r_y) requirement, likely because torsional stiffness of the column is implicitly considered when verifying $\alpha_m\alpha_s > 1.0$. The three axial force limits are also respected, which also suggests stable inelastic response. Design calculations based on NZS reveal that the cross section resistance is the dominant limit state. Verification of the SCWB in accordance with EC8 led to a larger value compared to those obtained from the other standards, mainly because axial load reduction is neglected in the calculation of the flexural capacity of the column. The interaction ratio for in-plane limit state is close to the S16 value; however, the interaction ratio for column lateral-torsional buckling exceeds 1.0, which suggests that limit state may occur under seismic load effects. When considering plastic design requirements, ψ for the condition in Figure 1c is -0.8 and this gives L_{Stable} = 3834 mm, which is less than the 4 m column unsupported length, also suggesting that out-of-plane instability is possible.

4 RESPONSE OF THE COLUMN STUDIED USING FINITE ELEMENT ANALYSIS

A finite element model of the selected W610x153 column (Figure 2a) was created in ABAQUS [18] based on the modeling approach discussed in Elkady and Lignos [5]. That model was calibrated and verified with experimental data from full-scale tests [6]. The column was modeled as fixed at the base with a nonlinear rotational spring at the upper end simulating the inelastic behaviour of the adjacent beams. The column was subjected to a constant axial compression equal to $0.2C_y$ and the symmetric cyclic lateral displacement history of AISC 341-10.

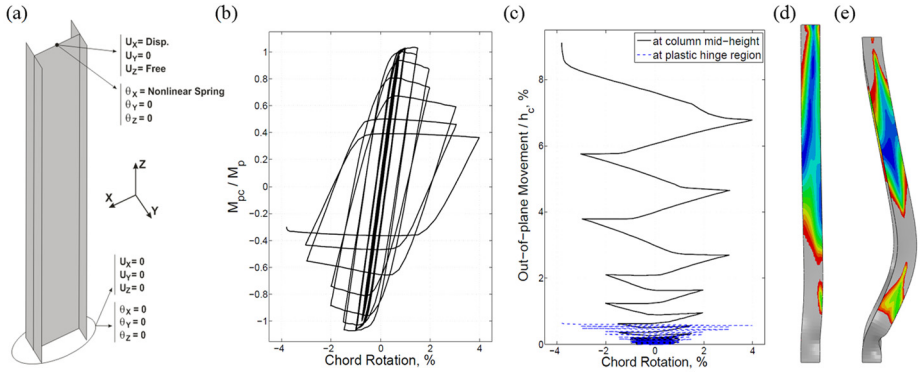

Figure 2. (a) Finite element model of the W610x153 column with boundary conditions; (b) Moment-rotation response of the column; (c) Out-of-plane deformations; (d) Out-of-plane deformations at the end of the second cycle of 1.5% storey drift; and (e) Out-of-plane deformations at the end of the first cycle of 4% storey drift.

Figure 2b shows the moment-chord rotation response of the column. Flexural yielding occurred near the base at the first cycle of 1.0% storey drift. At the first cycle of 1.5% storey drift, flange local buckling was initiated. In Figures 2c and 2d, column out-of-plane deformations initiated near the plastic hinge location at the second cycle of 1.5% drift amplitude due to flange local buckling occurring at this location. Out-of-plane deformations then propagated towards the mid-height as the applied drift ratio was increased. In Figure 2b, strength and stiffness started to deteriorate rapidly at the second cycle of 1.5% drift, as a result of the out-of-plane lateral deformations. Although, out-of-plane deformations were significant, very limited torsional deformations took place along the height of the member. At 4.0% storey drift, the flexural strength of the column reduced by more than 60% compared to the peak value due to the large out-of-plane deformations along with flange local buckling at the base. Limited out-of-plane deformation developed in the web at the plastic hinge region as a result of flange local buckling. Figure 2e shows the column deformed shape at the end of the first 4% storey drift cycle. As shown, lateral out-of-plane instability of the column eventually

occurred as a result of the large lateral deformations occurring along the member length. Three limit states were observed in the FE analysis: flexural yielding at the base, flange local buckling in the base hinge, and out-of-plane instability. The first two modes are expected and both are acceptable; however, the third one should not occur. Among the code provisions examined herein, it could be expected from the L_{cr}/r_y limit in CSA S16 and from both the interaction equation for lateral-torsional buckling and stability requirements for plastic design of EC3 (L_{Stable}). However, these provisions were not specifically developed to prevent out-of-plane instability triggered by local flange buckling in plastic hinges, as was observed in this analysis.

5 CONCLUSIONS

This paper describes the seismic design provisions for the first-storey columns with fixed based used in steel moment frames. The emphasis is put on stability requirements. The provisions of CSA S16, AISC 341 and AISC 360, NZS 3404, and EC3 and EC8 for columns were first reviewed. The design provisions were then illustrated for a deep slender steel I-shaped column. Finally, a detailed finite element model of the column studied was developed and analysed under constant axial compression load and cyclic lateral drift demand. The main findings can be summarized as follows:

- SCWB criterion in the design standards are very similar, although differences exist for the moment demands from the beams and for the column resistances.
- Verification of the column for member in-plane and out-of-plane instability under combined axial compression and bending moment is not explicitly required in Canadian and U.S. provisions. However, available interaction equations for elastic frames could be applied with adjustments for the anticipated inelastic response. Interaction equations of EC3 can also be applied to verify the member stability under combined axial and flexure demand. In the NZS standard, the member must be capable of developing its full plastic strength, without lateral-torsional buckling, and the axial load is limited to prevent in-plane stability due to the formation of a plastic hinge at the column base. All provisions predicted a stable in-plane response for the studied column, which was confirmed by the finite element analysis. However, none of these provisions except EC3 could predict the out-of-plane failure that was observed for the sample column examined in this study.
- Additional requirements included in the NZS 3404 to prevent instability in the base hinge region also failed to predict the observed out-of-plane instability for the deep slender column studied, although these provisions did correctly predict the behaviour of a column type I section also tested.
- Out-of-plane slenderness limit specified in CSA S16 and the minimum stable length requirement for plastic design in EC3 were not satisfied for the column studied. This suggests that those, or similar indicators could be used to prevent column out-of-plane instability. Further studies are however needed to develop criteria that better account for the interaction between local flange buckling and global instability for MRF columns with base plastic hinges.

These results highlight the vulnerability of beam type I section members to lateral instability if significant plastic hinging occurs during an earthquake. All the countries in this study implement capacity design procedures for multi-storey buildings to enforce strong column behaviour and NZS 3404 requires the implementation of a fixed base rotational stiffness limit of 1.67EI/L on column base modelling, to represent the flexibility inherent in a "fixed base" structure. These provisions are intended to suppress inelastic demand in the columns until the capacity design mechanism has formed in the MRF; in the 2010/2011 Christchurch earthquake series no inelastic action was observed in any columns of capacity designed steel frames [19].

REFERENCES

[1] Popov E.P., Bertero V.V., and Chandramouli S., "*Hysteretic behavior of steel columns*", Earthquake Engineering Research Center Report UCB/EERC-75-11, University of California, Berkeley, CA., 1975.

[2] MacRae G.A., "*The Seismic Response of Steel Frames*", University of Canterbury Research Report 90-6, University of Canterbury, Christchurch, New Zealand, 1990.

[3] Nakashima M., Takanashi K., and Kato H., "Tests of steel beam-columns subject to sidesway", *Journal of Structural Engineering*, *ASCE*, 116 (9), 2516-2531, 1990.

[4] FEMA., "*State of the art report on connection performance*", FEMA-355D, Federal Emergency Management Agency, Washington, D.C., 2000.

[5] Elkady A., and Lignos D.G., "Analytical investigation of the cyclic behavior and plastic hinge formation in deep wide-flange steel beam-columns", *Bulletin of Earthquake Engineering*, 13(4): 1097–1118, 2015.

[6] Elkady A., and Lignos D.G, "Dynamic Stability of Deep and Slender Wide-Flange Steel Columns - Full Scale Experiments", *Proceedings of the SSRC annual stability conference*, Orlando, FL, 2016.

[7] Uang C.-M., Ozkula G., and Harris J., "Observations from cyclic tests on deep, slender wide-flange structural steel beam-column members", *Proceedings of the SSRC annual stability conference*, Nashville, Tennessee, 17, 2015.

[8] Suzuki Y., and Lignos D.G. "Large scale collapse experiments of wide flange steel beam-columns." *8th International Conference on Behavior of Steel Structures in Seismic Areas (STESSA)*, Shanghai, China, 2015.

[9] CSA, *CSA S16-14*, *Design of Steel Structures*, Canadian Standards Association, Mississauga, ON, 2014.

[10] AISC, *ANSI/AISC 341-10, Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, IL, 2010.

[11] AISC, *ANSI/AISC 360-10, Specifications for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, IL, 2010.

[12] NZS, *Steel Structures Standard*, NZS 3404: Part 1:1997, Standards New Zealand, Wellington, New Zealand, 1997.

[13] CEN, *Eurocode 3 – Design of Steel Structures, DD EN-1993-1:2005,* E, Comite Europeen de Normalization, Brussels, Belgium, 2005.

[14] CEN, *Eurocode 8 – Design Provisions for Earthquake Resistant Structures, EN-1998-1:2003,* E, Comite Europeen de Normalization, Brussels, Belgium, 2003.

[15] Penga B.H.H., MacRae G. A., Walpole W.R., Moss P., Dhakal R.P., Clifton C., Hyland C., "Location of plastic hinges in axially loaded steel members", *Journal of Constructional Steel Research*, 64: 344-3516, 2008.

[16] Brownlee S.A., "*Axial Force and Plate Slenderness Effects on the Inelastic Behaviour of Structural Steel Beam-Columns*", the University of Canterbury, Christchurch, New Zealand, 1994.

[17] Elkady A., and Lignos D.G., "Effect of gravity framing on the overstrength and collapse capacity of steel frame buildings with perimeter special moment frames", *Earthquake Engineering & Structural Dynamics*, 44(8): 1289–1307, 2015.

[18] Simulia, "*Abaqus FEA*", www.simulia.com, 2011.

[19] MacRae G.A., Clifton G.C., Bruneau M., Kanvinde A. and Gardiner, S., "Lessons from Steel Structures in Christchurch Earthquakes", *Behaviour of Steel Structures in Seismic Areas STESSA 2015*, Shanghai, China, 2015.