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COMPARISON OF CANADIAN SEISMIC DESIGN PROVISIONS FOR TALL BRACED STEEL FRAMES IN HEAVY INDUSTRIAL APPLICATIONS

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Abstract: The seismic design provisions included in CSA S16-14 Annex M for tall steel CBFs for heavy industrial applications are reviewed and applied for a 65.4 m tall industrial structure located in Vancouver, BC. The design is compared with CBFs of the Moderately Ductile (Type MD) and Conventional Construction (Type CC) categories. Nonlinear response history analyses are carried out to examine the extent of brace inelastic responses and storey drifts over the frame height and the resulting axial load and flexural demands on the columns. For the structure studied, Annex M provisions leads to heavier and thus less economical structure. Frames designed in accordance with the three approaches could withstand the applied ground motions without collapse. Because of their high slenderness, braces in the three CBFs remained essentially elastic in tension, which contributed to the observed satisfactory response. Seismic load amplification as a function of structure height and column design moments as specified in Annex M could potentially be relaxed. Reduced damping effects should be considered for the design of tall Type CC CBFs.

1. INTRODUCTION

Steel structures used in large industrial facilities in the mining, oil and chemical sectors must be designed to withstand heavy production loads and the structural configuration is generally governed by the industrial processes and mechanical equipment. These facilities often require high-rise structures, without clearly defined floor levels and with marked geometric, stiffness and mass irregularities. To accommodate these geometrical constraints, achieve the required lateral stiffness and meet design standards, steel concentrically braced frames (CBFs) are typically used to resist lateral wind and earthquake loads.

As discussed by Rolfes and MacCrimmon (2007), seismic design rules in Canadian and U.S. codes have been developed primarily for conventional structures for commercial or institutional buildings. Consequently, these rules cannot be applied for heavy and irregular industrial steel frames. Industrial structures have different dynamic characteristics that may affect their seismic performance. They also possess lower damping and exhibit greater flexibility due to the absence of architectural elements. Their fundamental period can therefore be significantly longer than the values obtained from the empirical formulae specified in codes. Despite these differences, until recently, due to the lack of specific provisions for the seismic analysis and design of these structures, design engineers had no choice but to use the standard rules for the design of heavy industrial structures.

In Canada, a new Annex M was introduced in the CSA S16-14 steel design standard (CSA 2014), for the seismic design of heavy industrial steel structures. In this Annex, it is permitted to design elevated tanks, vessels, bins, or hoppers with symmetrically braced legs located in high seismic zones as Type MD (moderately ductile) CBFs for structure heights up to 60 m. This is 20 m higher than the limit specified for

Type MD CBFs used in conventional buildings. The provisions, however, require that seismic effects be determined from a dynamic (response spectrum) analysis and amplified to compensate for the reduced damping and limited redundancy of the structure. Cardoso and Coman (2015) examined the impact of Annex M on design forces and required steel tonnage for a complex 65.4 m tall industrial structure housing a vertical process. Structures located in Vancouver, BC, and Montreal, QC, were considered for two site categories. In each case, the design was compared to a simpler Type CC (Conventional Construction) CBF design which comes at the expense of larger seismic loads. For the structures in Vancouver (high seismic zones), the authors found that Annex M led to smaller brace forces but higher axial loads for beams and columns. In Annex M, columns must also be designed for a concomitant minimum flexural demand. Overall, slightly more steel was needed for Annex M compared to Type CC, mainly because of the column design provisions. The authors indicated that column design provisions in Annex M were probably too conservative and recommended that further study be performed to investigate the nonlinear seismic response of the two structure designs.

This article presents a preliminary study of the nonlinear seismic response of the structure studied by Cardoso and Coman. In view of the complexity of the original structure, the planar response of only one single-bay braced frame is examined herein. In addition to Annex M and Type CC designs, Type MD design without the special requirements of Annex M was also considered to evaluate the possibility of relaxing Annex M requirements. The structures were assumed to be located in Vancouver, BC, and all designs were performed in accordance to the 2015 National Building Code of Canada (NBCC).The seismic design provisions for the three design approaches are first summarized and compared. Application of the three design approaches to the studied braced frame is then illustrated and discussed. The results of nonlinear response history analyses performed on the three designs are presented and analyzed. Preliminary recommendations for possible simplifications of Annex M are formulated in the conclusions.

2. DESIGN METHODS FOR INDUSTRIAL STRUCTURES

For the design of CBFs for conventional building structures, the engineer can choose between Type MD, Type LD (Limited Ductility) and Type CC categories. The ductility-related force modification factor R_d for the three systems are respectively equal to 3.0, 2.0 and 1.5. For multi-storey buildings, seismic provisions for Type MD and Type LD are nearly the same, the main difference being that Type LD CBFs can be used for structures up to 60 m in view of the lower ductility demand expected because of the reduced R_d factor. In this study, only Type MD is considered as Type MD provisions formed the basis for the provisions of Annex M used to design steel CBFs for heavy industrial applications. So doing, impacts of the additional provisions of Annex M could be assessed more easily. The main seismic design requirements for Type MD, Annex M, and Type CC designs are summarized in Table 1.

As mentioned, the height limit prescribed for Type MD and Type CC CBFs is 40 m and the limit is increased to 60 m for CBFs designed in accordance with Annex M. The structure examined in this study is 65.4 m tall, which exceeds the limit for Types MD & CC systems while being close to the limit specified in Annex M. This allowed evaluating the appropriateness of the current limits for structures similar to the one considered herein. In the table, cross-out cells indicate provisions that were omitted in this study. All systems have the same overstrength-related force modification factor R_o. Seismic analysis for Type MD CBFs can be either the equivalent static force procedure (ESFP) or the dynamic response spectrum analysis (RSA), depending on the building height and period as well as structural irregularity conditions, as dictated in the National Building Code of Canada (NRCC 2015). The RSA method is prescribed for CBFs designed as Type CC or with Annex M. For consistency in this study, RSA was used in all three cases. For Types MD and CC frames, results from RSA must be multiplied by the ratio between the base shear from ESFP and the base shear from RSA when the former is larger. This is not required in Annex M; however, the design spectrum in Annex M must be amplified by a damping coefficient β to account for the higher seismic demand resulting from the lower damping of bare steel structures: ξ = 3% or 2% for bolted and welded structures, respectively. The structure studied here has bolted connections, which gives β = 1.227. In all three systems, the seismic loads must be amplified as a function of the height above a certain height. In this study, this amplification was not applied for Type MD and Annex M designs to examine if this requirement could be relaxed. The force amplification specified for Type CC was considered. In Annex M, the seismic design forces for CBFs taller than 40 m must be further amplified by a redundancy factor of 1.3 unless it can be demonstrated that failure of any brace or brace connection does not increase earthquake effects by more than 33% in the remaining members. The structure examined herein is a two-dimensional CBF with a simple X-bracing configuration and the 1.3 factor had to be applied.

Table 1 : CSA S16-14 seismic design provisions for steel CBFs in high seismic regions

Parameters	Type MD	Annex M	Type CC
Height limits (m)	40	60	40
Ductility-related factor (R_d)	3.0	3.0	1.5
Overstrength-related factor (R_o)	1.3	1.3	1.3
Analysis method	ESFP or RSA	RSA	RSA
Damping coefficient (β)	N/A	$(0.05/\xi)^{0.4}$	N/A
Base shear calibration	Yes	No	Yes
Amplified seismic forces	3% / metre above 32 m		2% / metre above 15 m
Redundancy factor	N/A	1.3 if h_n > 40 m	N/A
Brace width-to-thickness ratio limits for I-sections	KL/r ≤ 100: Class 1 KL/r = 200: Class 2 100 < KL/r ≤ 200: linear interpolation		Class 1 or 2
Brace probable resistances	$T_u = A_g R_y F_y$ $C_u = \min [T_u ; 1.2C_r / \phi]$ $C'_u = \min [0.2T_u; C_r / \phi]$, where C_r is computed using $R_y F_y$		N/A
Column design axial loads	Gravity plus forces from brace probable resistances		Gravity + 1.3E
Columns: additional requirements	Continuous & constant cross-section over a minimum of 2 storeys		N/A
	Beam-columns of class 1 or 2 sections with $M_f = 0.2M_{pc}$		Class 1 or 2 sections $M_f = 0$

Bracing members in CBFs designed as Type MD or following Annex M are expected to dissipate energy through inelastic buckling in compression and yielding in tension and capacity design is mandatory for these two structures. Brace sections must then satisfy stringent slenderness limits to delay local buckling and beams and columns must be designed to resist gravity loads plus forces induced by the braces reaching their probable resistances. Columns must also be designed as beam-columns with an accidental bending moment equal to 20% of the column plastic moment, M_{pc}, induced by non-uniform storey drift demands over the frame height. For the design of beams and columns, it is noted that brace induced forces may be limited to those producing a storey shear $V_{max,1.3}$ calculated with $R_d R_o$ = 1.3. This upper limit would generally control when oversized braces are selected to meet the applicable cross-section limitations or when T_u significantly exceeds the brace design loads as is the case for slender braces. For both systems, brace frame columns must be continuous and of same cross-section over a minimum of two storeys. As discussed later, this requirement could not be fully satisfied for the structure studied because of the tall storey height. For Type CC structures, the frame response is expected to remain essentially elastic due to the lower R_d value and seismic force amplification. Capacity design is therefore not required. Bracing members must still be Class 1 or 2 to prevent local buckling in case of brace buckling and columns must be designed for amplified seismic loads (1.3 E) to protect them against buckling in case of large seismic demand.

3. COMPARISON OF CANADIAN DESIGN METHODS

Figure 1a shows the prototype braced frame that was selected for this study. The seismic weight (W) and gravity loads (P) at each beam-to-column joints are given in the figure. Seismic weights at several levels exceed 150% of the weight at adjacent levels, which represents a structure mass (weight) irregularity as defined in the NBCC, a typical situation for such industrial applications. Braces, columns and beams are I-sections meeting the requirements of ASTM A992 (F_y = 345 MPa). The braces are positioned so that out-of-plane buckling takes place about their weak axis. Columns are oriented such that in-plane buckling is about weak axis. Additional detail of the complete structure can be found in Cardoso and Coman (2015).

Response spectrum analysis (RSA) of the frame was performed with the SAP2000 software (CSI 2017). For design, the braces' compressive resistances were computed with an effective length factor K = 0.5 (Tremblay et al. 2003) whereas K = 1.0 was used for the columns. For capacity design approaches, the brace's probable compressive resistances C_u and C'_u were determined considering K = 0.45 to reflect the actual buckling length between end connections. For all members, the lightest section satisfying all limit states and detailing requirements was selected.

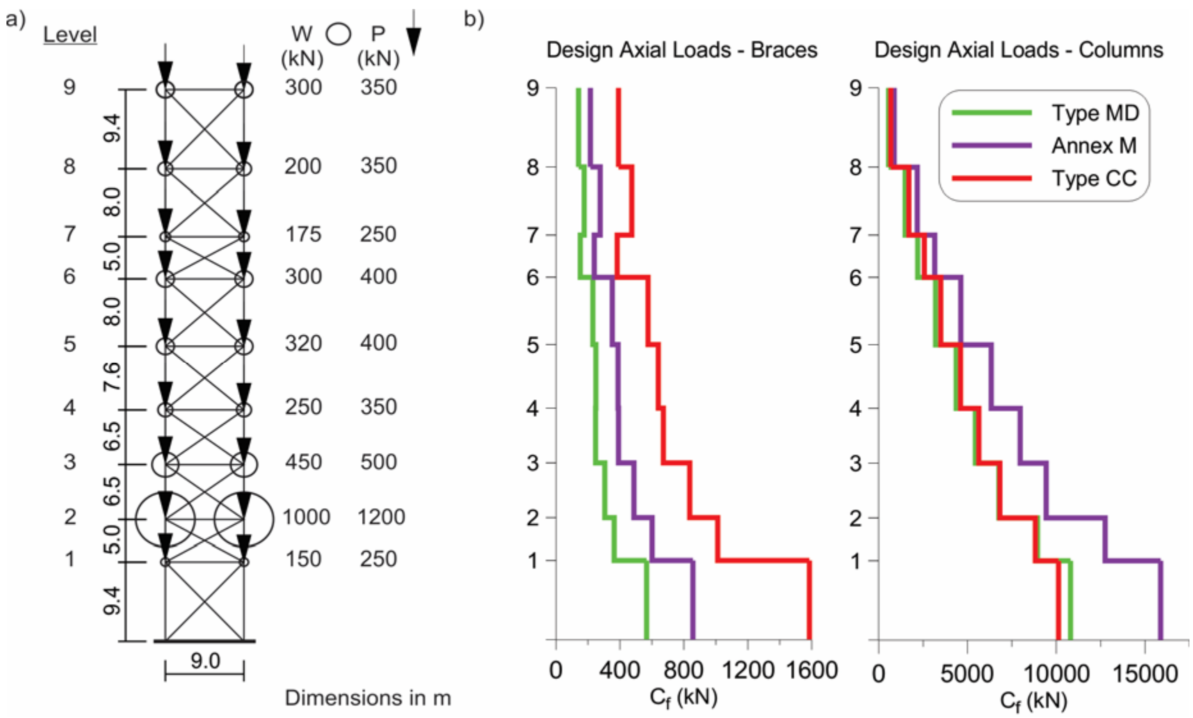

Figure 1 : Prototype structure: a) Geometry and properties; b) Design axial loads in braces and columns.

The factored compression loads C_f from gravity and earthquake loads are given in Figure 1b for the braces and columns of the three braced frame designs. Key design parameters and structure properties are given in the upper half of Table 2 for the three CBFs. As shown in Figure 1b, the lowest brace forces were obtained for the Type MD CBF. Amplification for damping (β = 1.227) and redundancy (1.3) prescribed in Annex M increased the brace seismic forces compared to Type MD. The required larger braces resulted in a stiffer structure with shorter fundamental period T_1 (Table 2), which also contributed to increasing the design brace forces. The final difference in design base shear V_d is given in Table 2 (983 vs 559 kN). It is noted that the period T_1 of The Type MD frame is shorter than the period T_a = 1.64 s given by the NBCC empirical formulae (T_a = 0.025 x h_n, where h_n = 65.4 m). Hence, adjustment of the RSA results based on base shear values was not needed for the Type MD frame and the reduction on design seismic loads that was expected for the Annex M frame because the Annex does not require scaling of the RSA results did not materialize for this structure. For the Type CC CBF, the amplification of the seismic loads for structures above 15 m reached the upper limit corresponding to R_dR_o = 1.3, as reflected by the design base shear $V_d = V_{max,1.3}$ = 1771 kN in Table 2. This resulted in the highest brace design loads among the three systems.

In Table 2, the base shears $V_{max,1.3}$ for the Type MD and Annex M CBFs are smaller than the base shears V_u obtained from the brace forces C_u and T_u. The same condition existed at every level of the two structures and reduced brace forces limited by $V_{max,1.3}$ were therefore considered when determining axial loads in beams and columns. In design, the brace compression force at a given level is set equal to C_u or C'_u, depending which one produces the most adverse effects, and the brace tension force is back-calculated from the storey shear $V_{max,1.3}$. In the Annex M frame, $V_{max,1.3}$ values were larger than in the Type MD CBF because of the shorter period and the seismic load amplifications due to damping and redundancy. This led to much higher column axial loads for Annex M compared to the Type MD frame. Column axial loads for the Type CC frame were similar to those obtained with for the Type MD CBF. As discussed, columns in Type MD and Annex M frames had to be designed as beam-columns with minimum flexural resistance, and the Annex M frame required the largest column sizes followed by the Type MD frame and, thirdly, the Type CC CBF. The larger columns of the Annex M frame contributed to the shorter period T_1 of that frame. Because of the tall storey heights in the structure, column sections were changed at every level, which did not comply with the column continuity requirement in CSA S16. As shown in Table 2, provisions for Types MD and CC CBFs led to similar amount of steel, the lighter braces in the Type MD design being counteracted by the smaller columns required for the Type CC CBF. The CBF designed following Annex M is the heaviest, mainly because of the much larger column sizes. The anticipated roof drift Δ_{Roof} is given in Table 2 as a function of the storey height h_s. As shown, low values are obtained for all three CBFs. The drifts are slightly larger for the Type CC frame compared to the Type MD and Annex M frames.

Table 2 : Design data and NLRHA results for the three CBF designs

		Type MD	Annex M	Type CC
Design	T_1 (s)	1.42	1.21	1.43
	V_d (kN)	559	983	1771
	$V_{max,1.3}$ (kN)	1689	2919	1771
	V_u (kN)	2470	3414	5309
	Δ_{Roof} (%h_s)	0.57	0.57	0.64
	Steel Tonnage (t): Braces	7.0	9.7	12.1
	Columns	37.6	52.4	29.0
	Total	48.4	66.6	45.2
NLRHA (MSD values)	T_1 (s)	1.38	1.16	1.38
	V_{max} (kN)	1711	2638	3317
	Δ_{Roof} (%h_s)	0.58	0.66	0.90
	Columns: C/C_f	0.94 - 1.22	0.79 - 0.98	1.11 - 1.22
	Columns: C/C_{prob}	0.58 - 0.67	0.49 - 0.54	0.89 - 1.03
	Columns: M/M_{pc}	0.02 - 0.09	0.02 - 0.11	0.03 - 0.10

4. NONLINEAR RESPONSE HISTORY ANALYSIS

The OpenSees computer program (Mazzoni, et al. 2006) was used to perform the nonlinear response history analyses of the CBF structures. To reproduce inelastic buckling in compression and yielding in tension, the columns and braces were modelled using the force-based nonlinear beam-column elements with fiber discretization of the cross-section, Steel02 material, co-rotational transformation, initial out-of-straightness, and residual stresses (Uriz 2005, Agüero et al. 2006, Lamarche and Tremblay 2011). The yield strength assigned to the Steel02 material was set equal to the probable yield strength R_yF_y = 385 MPa. Out-of-straightness having a half-sine wave profile with maximal amplitude of L/1000 was assigned in both directions. The residual stress pattern proposed by Galambos and Ketter (1959), with residual compressive stress up to $0.3F_y$, was employed. Beams were modelled using elastic beam-column elements.

Figure 2 shows details of the connections assumed for the frames. Columns were considered fixed at their bases to replicate the actual construction conditions. Column splices were modelled as fully rigid in flexure. As shown by Figure 2a, beam-to-column connections were bolted double angle shear connections with the angles extending the full depth of the beam plus gusset plates. The gusset plates were welded to the beams and the braces were connected to the gusset plates using end plate connections. For Types MD and Annex M CBFs, the details allowed for out-of-plane rotation upon brace buckling. The brace-to-beam connections were modelled as fully rigid in the plane of the frame. Out-of-plane rotational response of the brace-to-beam connections were modelled using a nonlinear spring (zeroLength) element with flexural strength and stiffness properties of the gusset plates, as recommended in previous studies (Aguerro et al. 2006, Hsiao et al. 2012). The stiffness provided by the bolted angle to the column was reproduced using a combination of zeroLength element and Pinching4 material calibrated with the test data from Stoakes and Fahnestock (2011), as depicted in Figure 2c. End plate connections were also used at the intersection of the bracing members. That connection was modelled as fully rigid in-plane and out-of-plane rotations.

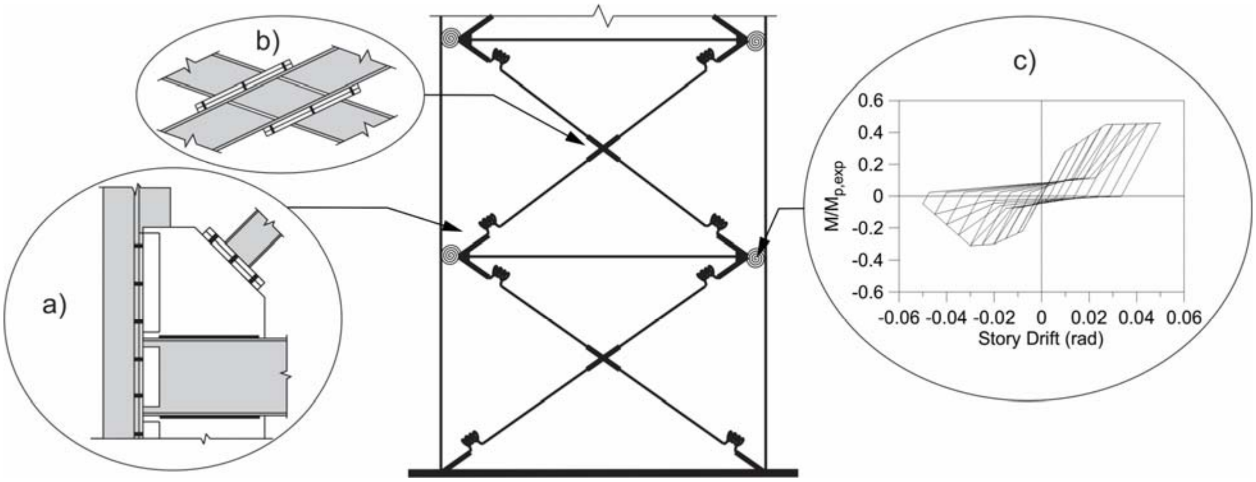

Figure 2 : Numerical one storey inelastic frame

Rayleigh damping was assigned with 3% of the critical damping value assigned in the first and third modes, consistent with the assumption made in Annex M. As specified in NBCC 2015, the structures were subjected to 3 suites of 5 ground motion records, one suite for each of the three earthquake sources contributing to the seismic hazard in Vancouver: shallow crustal earthquakes, subduction deep in-slab earthquakes and subduction interface earthquakes. The ground motions were selected from records from past earthquakes that matched the firm ground (site C) conditions. In the analyses, concomitant gravity loads shown in Figure 1 were first applied to the structure prior to the application of the ground motions, and global stability effects were included in the analyses.

5. NUMERICAL SIMULATION RESULTS

5.1 Storey Drift and Brace Force Responses

Figure 3a presents peak storey drifts expressed as a fraction of the storey heights, h_s, for the three CBF designs. Values from each individual record (Ind. Records) are given as well as the mean values of the five records producing the largest demands, Δ_{MSD}. The latter is considered herein as the seismic demand for the evaluation of the CBF designs, as specified in the NBCC. Maximum anticipated storey drifts from design (Δ_{max}) are also given for comparison. As shown, all three braced frames could withstand all ground motion records without collapse. Drifts also gradually increase towards the top of the structures, without significant deviations along the height of the frames, indicating that soft-storey response did not develop in the CBFs. For the Type MD and Annex M CBFs, this desirable response was obtained even if the provision for the amplification of the seismic loads as a function of the structure height was omitted in the design. This can be attributed to the fact that brace tension loads in these frames remained below T_u in all storeys, which contributed to the control of storey drifts. This aspect is discussed further below. For all frames, MSD drifts generally exceed the values predicted in design but all values remained well below the NBCC limit of 2.5% h_s. Peak roof drifts from NLRHA are given in Table 2. These values match well the design predictions for Type MD and Annex M designs. For Type CC, the NLRHA value exceeds the design estimate, likely because the frame response was elastic, as discussed below, and 5% damping was used in design.

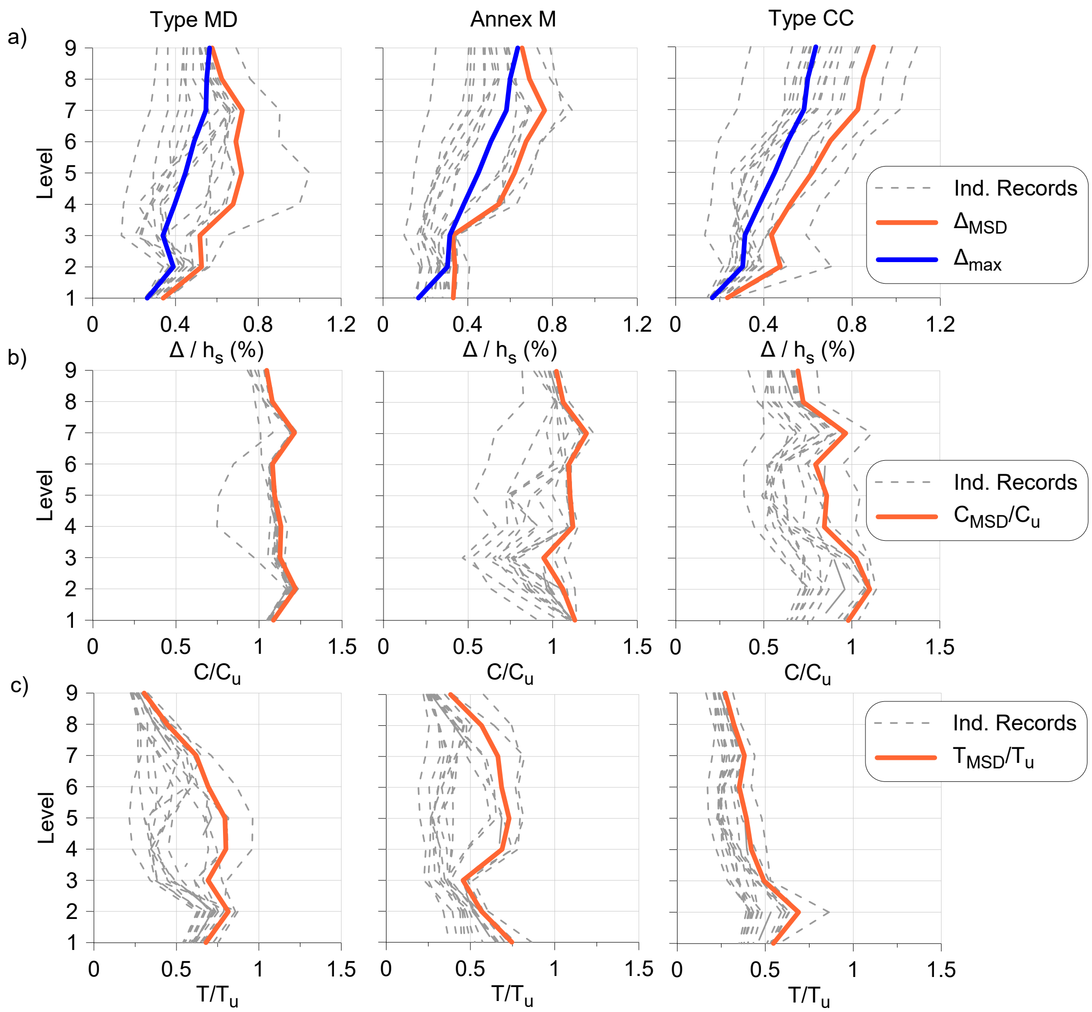

Figure 3: Peak response from NLRHA: a) Storey drifts; b) Brace compression loads; and c) Brace tension loads.

Figures 3b and 3c respectively present the peak compression and tension loads in the bracing members. The loads are normalized with respect to the corresponding brace probable resistances C_u and T_u. As expected, all braces of the Type MD and Annex M frames reached their compressive resistances and experienced buckling. As was also anticipated due to amplified design seismic loads corresponding to R_dR_o = 1.3, the braces in the Type CC frame remained essentially elastic, the peak compression force reaching C_u in only one level. Peak brace tension loads remained below T_u in all braces of the three frames, which means that brace tension yielding did not occur in these frames. The observed moderate demand-to-capacity ratios are attributed to the fact that the braces are quite long and slender in this structure due to the tall storey heights. This results in large differences between the brace resistances T_u and design compression forces C_f. These differences are more pronounced in the upper levels where braces are smaller and more slender, resulting in smaller T_{MSD}/T_u ratios. This behaviour is different from Type MD CBFs used in conventional building structures where braces are stockier and brace yielding in tension is generally expected. The low T_{MSD}/T_u ratios observed here justify the reduced brace loads based on $V_{max,1.3}$ when determining axial loads in beams and columns of the Type MD and Annex M CBFs. The reduction in brace tension forces is also due to the lengthening of the period of these two frames caused by brace buckling. MSD values of the peak base shear V_{max} from NLRHA are given in Table 2. These values are closed to the computed $V_{max,1.3}$, confirming the adequacy of the CSA S16 design approach. The value of V_{max} for the Type CC frame significantly exceeds $V_{max,1.3}$ in Table 2 because the frame response remained nearly elastic and $V_{max,1.3}$ was estimated assuming 5% damping.

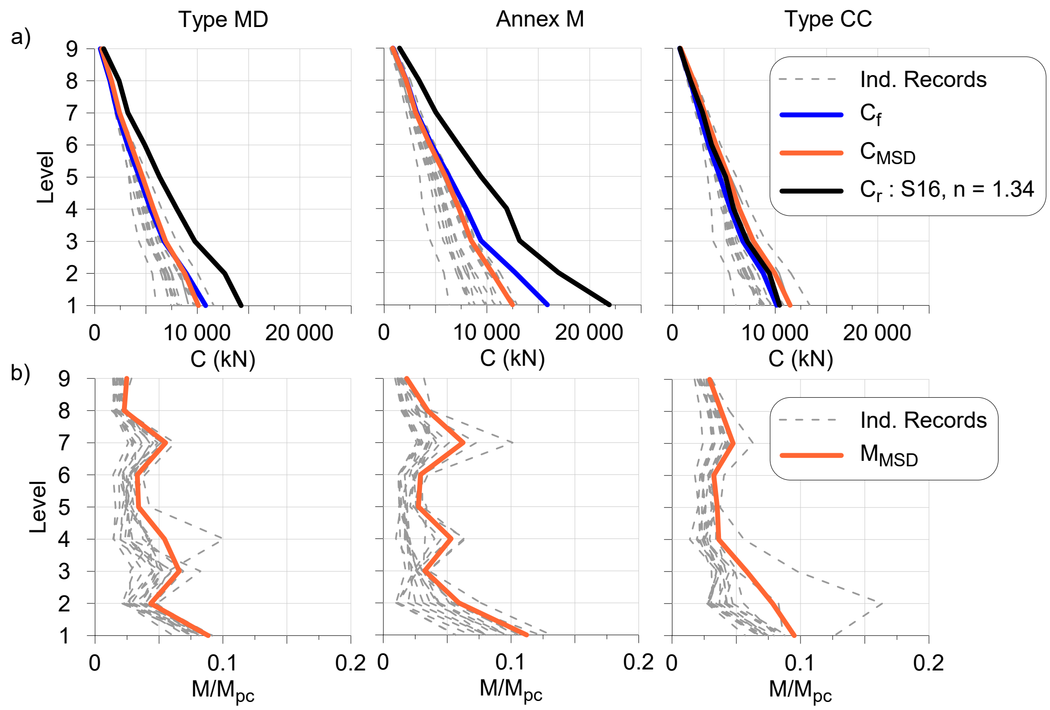

Figure 4: Peak demands on columns from NLRHA: a) Axial loads; b) Bending moments.

5.2 Columns Results

The MSD values for the column axial loads and column bending moments from NLRHA are displayed in Figure 4. In Figure 4a, the design factored loads C_f and column factored axial resistances C_r are also presented for comparison. For both Type MD and Annex M CBFs, the axial load demand was generally well predicted by the design methods used. Conversely, the simplified approach used for the Type CC CBF failed to adequately predict the axial load demand in the columns. Figure 5a presents the ratios between axial load demands and design factored loads C/C_f for the three frames. Ranges for this ratio are given in Table 2. The ratios for Type MD and Annex M frames follow the same trend over the height with a gradual increase over the frame height. NLRHA column axial loads in the Type MD frame in fact exceed the design predictions in most levels whereas those in the Annex M remained below the prediction. The shift between the two curves can be attributed to the fact that $V_{max,1.3}$ used in Annex M included the damping and redundancy amplification factors, which was not the case when following Type MD provisions. For these two frames, the variation of C/C_f over the height is due to the likelihood that all storeys above the level under consideration reach their storey shears $V_{max,1.3}$ reduces when moving towards the structure base. For Type CC, the ratio is more uniform over the frame height because the design values are obtained from response spectrum analysis rather than by cumulating brace induced forces from the structure's top. For this frame, underestimation of the design loads is attributed to the fact that the response spectrum analysis was performed assuming 5% damping. In CSA S16, the upper limit on storey shears $V_{max,1.3}$ that can be considered in capacity design assumes that the components being designed possess a dependable overstrength of 1.3. In the NLRHA, column bucking was not observed in the Types MD and CC CBFs because the columns were modelled using their probable resistances, i.e. using the probable steel yield strength R_yF_y = 385 MPa and omitting the resistance factor ϕ = 0.9, which increased the columns' strengths by 1.24 (= $R_yF_y/F_y/\phi$) compared to their factored resistances C_r. In addition, the column shapes were selected from a discrete list of available sections which provided further overstrength. The validity of this approach is confirmed in Figure 5b where the ratios between the axial load demand and the columns' probable resistances, C/C_{prob}, are less than or close to 1.0 for all CBFs. In this figure, the much larger margin for the Type MD and Annex M comes from the fact that columns of these frames were designed as beam-columns assuming a flexural bending moment of 0.2 M_{pc}. In all three cases, however, it is interesting to note that all three design procedures resulted in a uniform demand-to-capacity ratio over the frame height. To obtain a level of protection from Annex M comparable to that obtained with Type MD columns, the redundancy factor could be omitted in the calculation of $V_{max,1.3}$. This approach would be more consistent with the objective of the redundancy factor of Annex M which is to reduce the likelihood of a brace fracture or failure of a brace connection in a less redundant frame, rather than preventing column failure. If the frame was to be designed as a Type CC CBF, it would be desirable to have greater safety against column buckling than shown in Figure 5b. This could be achieved by applying the damping amplification factor to the seismic demand obtained from RSA to obtain more realistic axial load demands for column design.

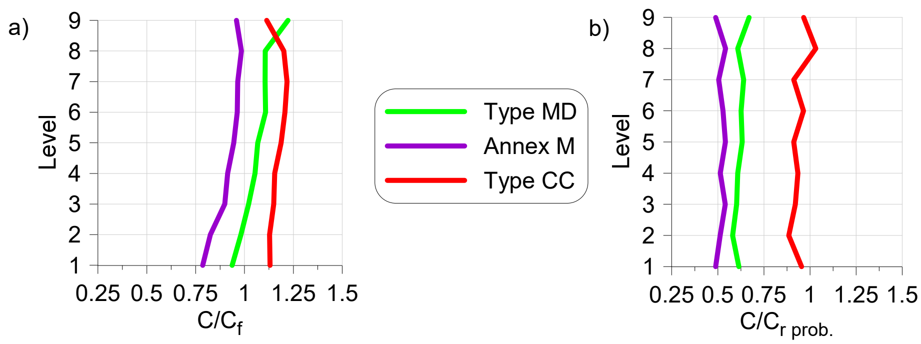

Figure 5: Comparison of axial load demands in columns with: a) design factored loads; and b) columns' probable resistances.

Figure 4b shows the flexural demand in the columns normalized to the columns' plastic moments, M/M_{pc}, and Table 2 gives the ranges of the plotted values. As shown, the flexural demand on the columns of all frames is very similar: it gradually increases from the top to the base of the structures, reaching a maximum value of approximately 0.1 M_{pc} in the first level. This maximum value is less than 0.2 M_{pc} used in the design of the columns of the Type MD and Annex M frames. For these two frames, these limited moments are probably due to the uniform drift response that was achieved because none of the braces yielded under the ground motions. For the Type CC frames, the low bending moment demand in the columns can also be attributed to the frame elastic response. However, considering that the columns possess limited margin for axial compression, it would be advisable that the observed flexural demand be considered if Type CC CBFs were to be used in tall heavy industrial structures.

6. CONCLUSION

The seismic response of a steel concentrically braced frame (CBF) part of an industrial structure 65.4 m tall located in Vancouver, BC, was evaluated through nonlinear response history analyses. Three different designs available in CSA S16-14 were examined: assuming the CBF was of the Type MD and Type CC categories, and using Annex M for heavy industrial structures. These approaches were followed without consideration of the frame height limits currently specified in codes. Furthermore, the seismic load amplification as a function of the structure height specified for Type MD and Annex M CBFs was omitted in design. The main conclusions of the study can be summarized as follows:

- Annex M resulted in the heaviest CBF with a total steel tonnage of 66.6 t. The Type MD and Type CC CBFs required 48.4 and 45.2 t of steel, respectively. The heavier Annex M design was attributed to its shorter fundamental period and the stringent design requirements for the columns.
- All three frames could withstand the applied ground motions without collapse. They all exhibited limited and uniformly distributed storey drift demands over their heights, which is mainly attributed to the fact that all braces of the three frames remained elastic in tension because of their high global slenderness and large overstrength in tension.

- For CBFs designed in accordance with Annex M, the redundancy factor need not be applied when determining the structure drifts and the upper limit on storey shear $V_{max,1.3}$. In frames where $V_{max,1.3}$ governed at every level, the seismic load amplification as a function of the structure height could be omitted and the design moments for the columns could be reduced to 0.1 M_{pc}.
- Type CC CBFs frames can exhibit satisfactory response for industrial structures up to 65 m in height. For such applications, it is recommended that the seismic loads be amplified to account for the reduced damping and that columns be designed for a minimum bending moment of 0.1 M_{pc}.

This study was limited to a simple planar braced frame. Further investigation is needed on other industrial structures having different geometries and mass distributions to confirm the findings of this study.

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