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INVERTED-V (CHEVRON) CONCENTRICALLY BRACED FRAMES – COMPARATIVE STUDY AND VERIFICATION ANALYSIS

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ABSTRACT

The following paper presents the results and discussions of a comparative study of inverted-v (chevron) concentrically braced frames (Iv-CBFs) designed according to two different seismic codes, namely Eurocode 8 (CEN 2005) and New Zealand Standard NZS 3404:Part 1 (1997), abbreviated as EC8 and NZS respectively throughout the remainder of this paper. The design of the Iv-CBFs will be completed for three different reference buildings, namely a three, six and twelve storey building.

Generally, as will be shown in this paper, designs completed with EC8 result in lower base shears compared to NZS design. This is attributed primarily to two factors, that is the scaling factor C_s and the lower permitted ductility limits of NZS over EC8. As will be seen, the results of nonlinear time history analyses (NLTHA), completed using the finite element software SeismoStruct (SeismoSoft 2014), show that the EC8 design procedure may lead to some early brace fracture in some cases leading to soft storey failure, whilst the NZS method tends to over-predict base shears. Of concern are the code estimates of inter-storey drift and overall storey displacements compared to the results of the NLTH analysis.

Keywords: Braced frame; Chevron; Steel;

1. INTRODUCTION

Concentrically braced frames (CBFs) provide an economical and attractive alternative to traditional steel framing for both new building design and retrofit of existing structures. For areas of low seismicity, tension-only CBFs are common, but tension and compression braces are also seen. However, tension and compression braces are most typically used in areas of medium to high seismicity due to their better performance and hysteretic behaviour. Brace configurations are typically X-braced or single diagonal braces grouped in even pairs and positioned in opposite orientations, refer Figure 1a.

As a subset of CBFs, inverted-v concentrically braced frames (Iv-CBFs) or chevron frames, can provide similar benefits compared to traditional CBFs but can also accommodate the benefit of fitting both braces into a single bay of a structure (similar to X-braced CBFs) but also potentially maintain access (or openings) within the bay. This can be extremely advantageous in seismic retrofit strategies where placement of braced frames can be quite limited due to pre-existing building layouts as well as access requirement for new foundations which will be required for any new strengthening schemes.

Of course, similar retrofit strategies can be achieved using steel moment resisting frames (MRFs) or eccentrically braced frames (EBFs). However, it is often found, that the sizes of these frame members have to become quite large compared to CBFs in order to provide the increased stiffness necessary to meet displacement compatibility requirements of the structure being retrofitted. In most cases, the structure being retrofitted will consist of masonry (or infill masonry) and/or reinforced concrete wall or frame structures. Iv-CBFs provide an excellent alternative in these cases without the need to explore more expensive, but generally better performing, systems such as buckling restrained braces (BRBs) for example.

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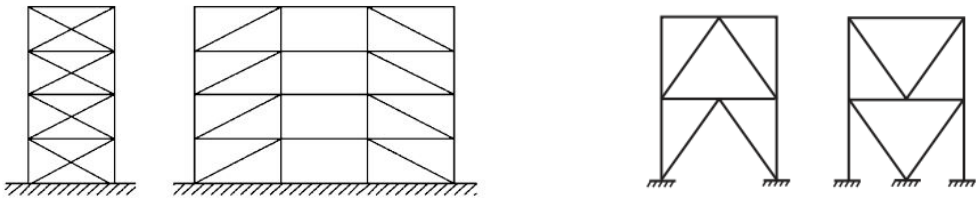


(a) (b)

Figure 1. Typical concentric braced frames, (a) concentric bracing and (b) V-bracings

This paper now proceeds to compare two current approaches for designing Iv-CBFs using European and New Zealand standards and guidelines. The differences in design procedures are discussed with emphasis on the restrictions on allowable ductility, brace slenderness and building height. In the New Zealand context, the seismic amplification factors used to increase the seismic design coefficients and consequently seismic demands for braced framed structures will be discussed. The merits of these design constraints will then be verified using a series of nonlinear time history (NLTH) analyses models using the commercial software (SeismoSoft 2014).
To obtain a set of comparable results for each code, noting the differing material grades and section sizes available for each region, material properties and section libraries will be consistent with that of European standards. Further to this, the lateral forces from the seismic analysis will be that which results from a modal response spectrum analysis (MRSA) in line with EC8. Again, each code has different approaches to deriving the lateral demands but as only the prescriptive design procedures are of interest in this comparative study, similar lateral forces will be used for both sets of designs.

2. COMPARATIVE STUDY

2.1 Background

A comparative study of the design of inverted-V (chevron) concentrically braced frames (Iv-CBFs) has been undertaken. This study compares the design philosophies and results between European and New Zealand standards. This study attempts to compare only the design methods and limitations of each code. To that end, consistent material properties and similar steel section libraries will be used regardless of what design code is used. The most noticeable differences being that cold formed hollow sections are typical in New Zealand/Australia, while hot rolled hollow sections are typical in Europe. Further to this, New Zealand rolled section sizes (most notably for UB sections) are limited to a maximum of 610UBs compared to the larger rolled section size UBs available in Europe.
Therefore, to have a set of comparable results for each code, material properties and section libraries will be consistent with that of European standards. Further to this, the lateral forces from the seismic analysis will be that which results from the modal response spectrum (MRS) analysis method in line with Eurocode 8 (EC8) (CEN 2004).

2.2 Seismic demands

The MRSA method as outlined in cl.4.3.3.3 of EC8 has been used to determine the seismic demands for this comparative study. Within EC8 a MRS analysis can be completed independently of a lateral force method of analysis (LFMA). This differs from a MRS analysis carried out in conjunction with the NZS seismic loading standard. Where NZS allows the use of the MRS method but the results must be compared to the equivalent static method (ESM) or (LFMA under EC8 nomenclature) and scaled accordingly. Depending on the ratio V/Ve it may be necessary to scale up V. Where V is the base shear resulting from a MRS analysis and Ve is the base shear resulting from the ESM. If V is less than 80% of Ve for regular structures or less than 100% of Ve for irregular structures, then the base shear, V resulting from the MRS analysis must be scaled up by the appropriate factor k according to Equation 1a or 1b for a regular or irregular structure respectively.

$$k = 0.8\ V_e/V \tag{1a}$$
$$k = V_e/V \tag{1b}$$

This seems nonsensical compared to EC8 as effectively what NZS 1170.5 is saying is that you are not permitted to use the ESM on irregular structures (as does most codes including EC8) but should in fact use a MRS analysis and then compare the results (and even scale up) to the magnitude of the ESM results. To further differentiate EC8 from this, EC8 allows the use of a base shear correction factor, γ for use with the LFMA (or ESM) for buildings with two or more storeys and where T1 ≤ 2TC (cl.4.3.3.2.2(1)P of EC8). Where it is acknowledged that without this correction factor, for higher mode effects and irregularities, the LFMA would yield conservative base shears.
Note: for the purposes of this study only the differences in design are of interest and will be compared. Obviously a large difference would result in the size of the members if two different base shears were used. This would only aid in distorting the results of the true comparative study. Therefore only a single base shear will be used for the design of members for both EC8 and NZS CBFs. This base shear will be that which results from a MRS analysis using EC8.

2.3 NZS Amplification Factor

EC8 places no additional seismic amplifications on the structure outside the demands obtained from the MRS analysis for the site in question.
New Zealand steel code, NZS 3404:Part 1 (1997) and commentary to the steel code, NZS 3404:Part 2 (1997), does however place additional seismic amplifications on CBFs and EBFs in the form of a multiplying coefficient. These coefficients, named Cs factors, vary from 1.0 to 2.1 depending on the slenderness of the bracing members, the ductility of the system and the number of storeys of the structure. Further height limitations are also incurred depending on the type of CBF system employed (Iv-CBFs or X-braced CBFs) and whether the CBFs forms part of an independent or dual system. Where Iv-CBFs have more severe height limitations compared to X-braced CBFs and independent structural systems enforce larger Cs factors compared to dual systems. Refer to Figure 2 for a typical example of Cs factors used for category 1 systems for CBFs.

Table 12.12.3(1) – C_s factors for Category 1 CBFs

Category 1 systems	**Compression brace slenderness ratio** $\frac{k_e L}{r}\sqrt{\left(\frac{f_y}{250}\right)}$		
Number of storeys	**≤ 30**	**≤ 80**	**≤ 120**
1	1.0	1.3	1.6
2 – 3	1.1	1.45	1.75
4 – 5	1.2	1.55	1.9
6 – 8	1.3	1.7	2.1

Figure 2. NZS scaling (C_s) factors

The values of C_s presented in NZS 3404:Part 1 (1997) tables 12.12.3 (Figure 2 above) are derived from three key variables as follows:

- Inelastic demand on the system
- The slenderness ratio of the braces
- The number of storeys

Analytically speaking, the C_s factor is a function of two variables, A and B, depending on the category of the system. Further information on this can be found in section C12.12.3(iv) of NZS 3404:Part 2 (1997).
The variable A accounts for the deterioration in inelastic performance with increasing brace slenderness. Based on the work completed by Remennikou and Clifton (1997), a value for the variable A can be

reasonably determined by Equation 2.

$$A = {}^{1}\!/_{[0.5(1+\alpha'_c)]} \tag{2}$$

Where α'_c is the post-buckling capacity factor. A full definition of the variable A can be obtained in section C12.12.3(i) of NZS 3404:Part 2 (1997). The Variable B accounts for the departure of the CBF system from the preferable o-type behaviour mechanism (weak beam – strong column) towards the less desirable s-type behaviour (strong beam – weak column). Further information on this can be found in C12.12.3(iii) of NZS 3404:Part 2 (1997).
It is worth noting that many structures will rarely be independent as lift/stair cores are normally be made of reinforced concrete wall elements and will contribute to the lateral load resisting system. For the case of this comparative study it is assumed that an independent system exists.
With reference to above, C_s factors will not necessarily be treated as purely a demand amplification. They are more a means (albeit a conservative one in the authors view) to control undesirable soft storey mechanisms forming in the structure. As a result, the C_s factors will be applied to the design of the CBFs for the NZS design case and included in this comparative study.

2.4 Building Performance Factors and Ductility

EC8 uses a building behaviour factor, q to reduce elastic seismic demands to account for inelastic response of the building structure. This factor is primarily related to ductility but also allows for building behaviour factors related to the type of building material and structure type. EC8 is also divided into different levels of ductility performance classes namely; low (DCL), medium (DCM) and high (DCH). For Iv-CBFs, EC8 places an upper limit on q equal to 2.5.
NZS uses a similar method albeit the factor is separated into two distinct values, a building performance factor, S_p and the ductility factor, k_μ. The ratio of which would be analogous to the behaviour factor, q from EC8 albeit it is acknowledged that EC8 does not explicitly have an equivalent to the S_p factor, refer Equation 3. The building performance factor, S_p is a simple scaling factor which tries to take account of factors that would not necessarily be easily captured in an analysis. For example:

- Calculated loads correspond to peak accelerations which happens only once and is unlikely to lead to significant damage,
- Structural elements are typically stronger than predicted due to higher material strengths, strain hardening and strain rate effects etc.,
- Structural capacity is typically higher than predicted due to non-structural elements and redundancy,
- Higher damping is normally present through non-structural elements and foundations.

$$q \cong k_\mu / S_p \tag{3}$$

The ductility factor, k_μ in NZS 1170.5 accounts for the actual ductility assumed for the system depending on the period of the structure. There is no limitation on the level of ductility that can be assumed for Iv-CBFs according to NZS 3404. However, the higher the assumed ductility the more severe the penalties, refer the discussion regarding the C_s factor in section 2.3.

2.5 Brace Post Buckling Capacity

Both EC8 and NZS assume a post buckled brace capacity for use in tension and compression systems. This is particularly relevant for Iv-CBFs as lateral resistance is taken up by the in-plane bending capacity of the beams upon buckling of the compression brace. In order to determine the design forces on the beam under earthquake loading an estimate of the post buckling capacity of the compression brace is required. The additional force applied to the beam is then the difference between the vertical component on the tension brace and the vertical post buckled capacity of the compression brace. There is obviously no net vertical force prior to brace buckling as the two braces cancel each other out.

Calculating the post buckling (and even the buckling) capacity of a brace is not straight forward and will vary from one code to another. In terms of calculating the post buckling capacity of a brace, numerous studies (Wijesundara, Bolognini et al. (2009), Tremblay (2002), Goggins, Broderick et al. (2005b)) have developed equations based on the brace slenderness and the assumed ductility demand.
Despite these works, section 6.7.4(2) NOTE 1 of EC8 recommends a simple expression to calculate the post buckling capacity of the brace, $\gamma_{pb}N_{pl,Rd}$, where the recommended value for γ_{pb} is 0.3 and $N_{pl,Rd}$ is the compression capacity of the brace. This is quite a low value to use but is somewhat justified for the following two reasons:

- Larger slenderness values are allowed for compression braces in EC8 compared to NZS and as a result a larger amount of braces will have much lower post buckling capacities and the value of 0.3 may be justified to some extent as an average of the complete steel section list;
- Assuming a lower value of γ_{pb} means a larger net difference between the vertical component of the tension and post buckled brace capacity and hence a larger more conservative beam design.

NZS 3404 on the other hand allows for the calculation of the brace post buckling capacity based on Equations C12.2.3(1) and C12.2.3(2) of NZS 3404:Part 2 (1997) which are taken from the work of Remennikou and Clifton (1997) and are presented here as Equations 4a and 4b.

$$\alpha'_c = {}^{42.15}\!/_{\lambda_n^{1.1}} \leq 1.0 \quad \text{for braces of category 1 and 2 systems} \qquad (4a)$$

$$\alpha'_c = {}^{7.7}\!/_{\lambda_n^{0.6}} \leq 1.0 \quad \text{for braces of category 3 systems} \qquad (4b)$$

In these expressions, α'_c equates to γ_{pb} in EC8 and the post buckling capacity of the brace is then $\alpha'_c N^{oc}_{brace}$. Where λ_n in Equations 4a and 4b is the modified member (brace) slenderness in accordance with section 6.3.3 of NZS 3404:Part 1 (1997) and N^{oc}_{brace} is the brace over-strength compression capacity.

2.6 Building Displacements and Inter-Storey Drifts

To the authors knowledge, no additional displacement or drift requirements for Iv-CBFs are discussed in EC8 or other related guidance. Displacements induced by seismic actions are discussed in section 4.3.4 of CEN (2004) and equation 4.23 of this clause is repeated here as Equation 5.

$$d_s = q_d d_e \qquad (5)$$

Where, d_s is equal to the displacement of a point of the structural system induced by the design seismic action, q_d is the displacement behaviour factor, assumed equal to q unless otherwise specified and d_e is the displacement of the same point of the structural system, as determined by a linear analysis based on the design response spectrum.
Note: Second order (P-Δ) effects do not need to be considered if the inter-storey drift sensitivity coefficient θ is less than 10% such that:

$$\theta = {}^{P_{tot} d_r}\!/_{V_{tot} h} \leq 0.10 \qquad (6)$$

Where P_{tot} is the total gravity load at and above the storey under consideration, d_r is the design inter-storey drift, V_{tot} is the total seismic storey shear and h is the inter-storey height. The value of θ should not exceed 0.3. If theta is between 0.1 and 0.2, the second-order effects may be approximately taken into account by multiplying the relevant seismic action effects by a factor of 1/(1-θ).
Similar means of calculating the design seismic displacement is inherent within NZS 1170.5 (2004) with some subtle differences. These differences will not be discussed here except that some additional

requirements are necessary for calculating the inter-storey drift of any structure and further amplifications of the displacement demand depending on the structural system employed. For Iv-CBFs, a scaling factor equal to two is required when calculating the seismic design displacement of the structure. The rational for this is because the inelastic behaviour of an Iv-CBF is largely controlled by beam in-plane defection not unlike a D-type eccentrically braced frame (EBF) where the beam is expected to yield. As the displacement of the structure is determined from an elastic analysis where compression braces are assumed unbuckled, some amplification of the structural displacement appears appropriate.
Further to these requirements, NZS 1170.5 (2004) (New Zealand loading code) requires an amplification of the inter-storey drift depending on the height of the structure. This is achieved by the drift modification factor, k_{dm} as shown in Table 7.1 of NZS 1170.5 (2004) and repeated here as Table 1.

Table 1. NZS drift modification factor.

Structure Height	Drift modification factor, k_{dm}
h < 15 m	1.2
15 ≤ h ≤ 30 m	1.2 + 0.02(h - 15)
h > 30 m	1.5

Therefore, compared to EC8, and assuming the elastic displacements are equal, an NZS design could have a drift in the order of two times k_{dm} equal to 2.4.

2.7 Verification of Design Results

Section 6.7.2(3) of CEN (2004) (EC8) requires that any ductile CBF system with both the tension and compression braces utilised in resisting the seismic demand must be verified via the following:

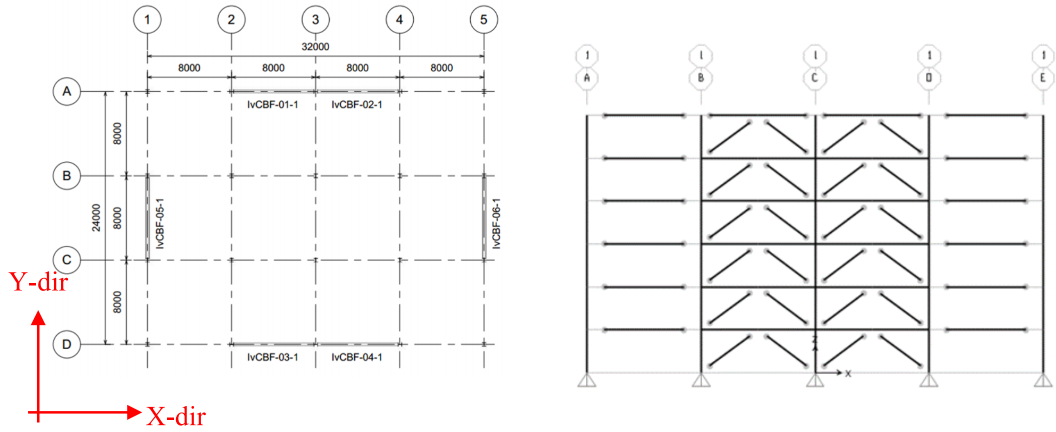
- A nonlinear static (pushover) global analysis or nonlinear time history analysis;
- Both pre-buckling and post-buckling situations are taken into account in the modelling of the behaviour of the diagonals and;
- Background information justifying the model used to represent the behaviour of diagonals is provided.

The latter requirement would typically include material test coupons to verify the material strengths of the nonlinear model and possibly geometric imperfections used to assume realistic brace buckling characteristics.
No special verification procedures are required for CBFs of any kind under NZS criteria.

3. NUMERICAL STUDY

3.1 Reference Buildings

Three reference buildings have been selected to complete the comparative study. These consist of three, six and twelve storeys respectively. Each building is regular in both plan and elevation with a consistent floor plan of 3 x 4 bays of 8.0 m in each direction. Refer to Figure 3 for the reference ground floor plan of all buildings and also the x-direction elevation for the six storey building.
Due to page limitations, only the results in the building x-direction are presented with focus on global building responses such as storey shears and inter-storey drifts. Results determined from the MRS analysis for each code are compared against each other and also against the results of a NLTH analysis. In the analysis and results presented in the following sections, only the ultimate limit state ULS (no-collapse in EC8) is considered. In a real design, member and global results should also conform to the requirements of a serviceability limit state (damage limitation in EC8). Results and details of the damage limitation checks will be presented in a future paper.



(a) (b)

Figure 3. Reference building (a) typical floor plan and (b) x-dir brace layout for 6-storey building

For both the MRS and NLTH analyses, seismic demands were derived for a fictitious site with the following characteristics:

Response spectra	Type 1
Soil type	D
Reference peak ground acceleration, a_{gR}	0.25
Importance factor, γ_I	1.0 (Importance class II)

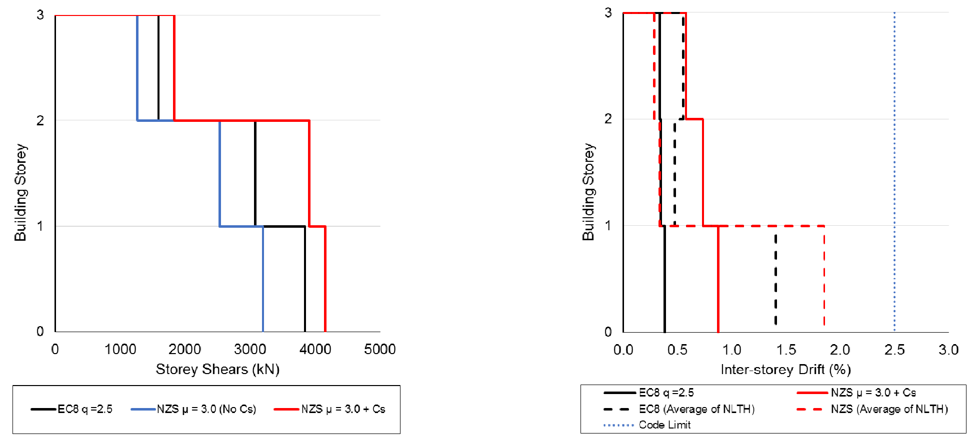
The discussion will begin by comparing the MRS analysis results for EC8 and NZS designs. The comparison to the NLTH analysis will then proceed this discussion. It should also be noted that in order to limit this study to examining the design procedures for each code, only section sizes available in the UK (at the time writing) are included in this study. Further to this, only square hollow sections (SHS) will be adopted for the seismic design of the brace members.

3.2 Results of Modal Response Spectrum Analysis

As shown in Figure 4 to Figure 6, the seismic demands for the NZS case are larger for all three reference buildings. As discussed in section 2, this is generally attributed to the amplification factor, C_s. For the three storey building, it is visible that the EC8 demands exceed the NZS demands when no C_s factor is used. However, as anticipated, the EC8 demands are less than that of NZS while including the C_s factor. The larger reduction factor (S_p/k_μ) used in the NZS design, compared to q in EC8, is the controlling factor why the NZS demands (with no C_s factor) is the lesser of the two codes.

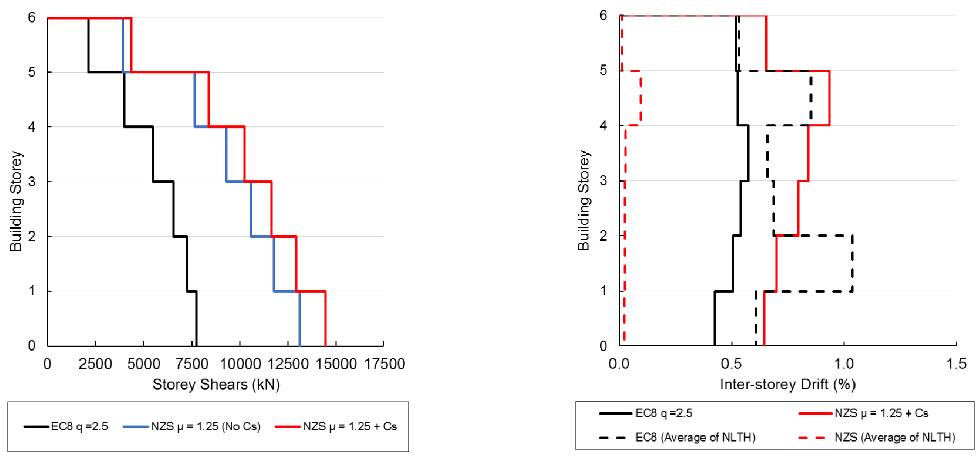
For the six storey reference building, the EC8 design results in smaller storey forces and displacements compared to the NZS designs with and without the C_s factor. It is also noteworthy, that for the six storey reference building, NZS reduces the allowable ductility capacity used in design due to storey height. In this instance, the building has been designed with nominal ductility compared to EC8 which still utilizes the full code allowance of q equal to 2.5 for Iv-CBFs.

Due to the height of the 12 storey building, NZS requires the building to be designed as fully elastic. Hence the C_s factor in this case is equal to 1.0. As expected, EC8 still allows the use of q equal to 2.5 and hence results in substantially less building demands compared to the NZS design. This results in NZS having nearly double the base shear as EC8. Of interest is the large increase in drift above level 3 for the NZS design. This is a consequence of a vertical irregularity in the seismic resisting system for the NZS case. Due to the larger storey shear forces, and keeping within the standard SHS profiles available, two additional braced bays were required for level 1 to 3 for the NZS design. This results in a vertical irregularity and the seismic drifts as shown. The maximum drifts for each building, in the x-direction, measure 0.75% and 2.21% for the EC8 and NZS designs respectively.



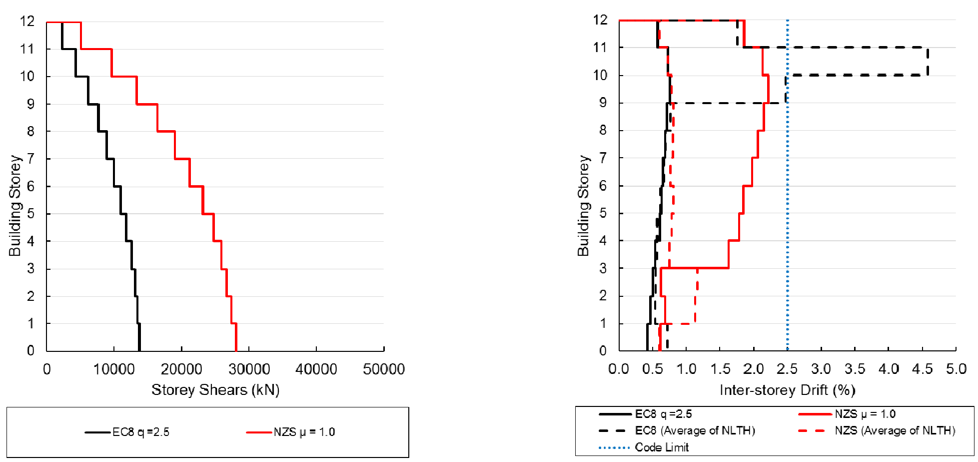
(a) (b)

Figure 4. Comparison of (a) storey shears and (b) inter-storey drifts for EC8 and NZS



(a) (b)

Figure 5. Comparison of (a) storey shears and (b) inter-storey drifts for MRSA for EC8 and NZS



(a) (b)

Figure 6. Comparison of (a) storey shears and (b) inter-storey drifts for MRSA for EC8 and NZS

In this section, the results of the three, six and twelve storey buildings have been presented. In general, the EC8 design procedure produces the lowest storey shears. This is because EC8 allows higher ductility levels compared to NZS for Iv-CBF structures with more than four stories typically. For structures of less than four stories, ductility levels are similar for each code if not greater for the NZS case. Further to this, NZS employs a load amplification factor, C_s, as a method to control undesirable failure mechanisms throughout the height of the structure which further increases the base shear.

Table 2. Seismic weight, building periods, performance factors and base shears (x-direction).

Reference Building	Seismic Weight	Building Period		Performance factor		Base shear		Peak Drift	
	(kN)	**EC8** *(s)*	**NZS** *(s)*	**EC8** *(q)*	**NZS** *(k_μ/S_p)*	**EC8** *(kN)*	**NZS** *(kN)*	**EC8** *(%)*	**NZS** *(%)*
3-storey	13,050	0.34	0.32	2.5	3.06	3,840	4,153	0.39	0.87
6-storey	27,860	0.56	0.54	2.5	1.33	7,742	13,138	0.57	0.94
12-storey	56,030	0.87	0.86	2.5	1.11	13,774	28,058	0.75	2.21

Table 3. Member sizes for three storey building.

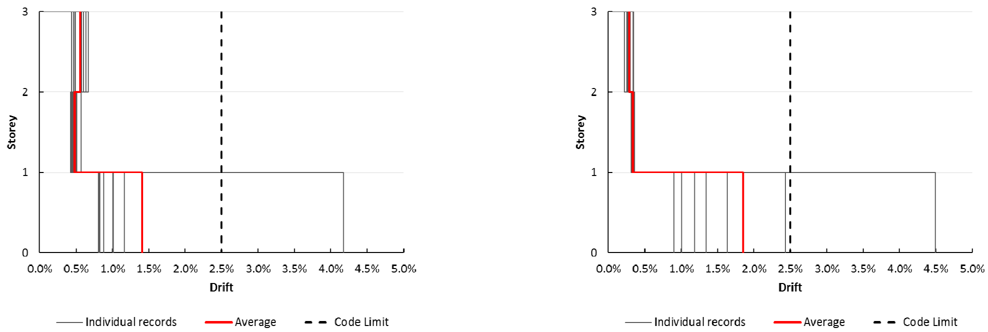
		Eurocode 3 and 8		NZS 3404	
Level	**Member**	**Section**	***D/C Ratio***	**Section**	***D/C Ratio***
3	Brace	120x120x4 SHS	0.95	120x120x6.3 SHS	0.87
	Beam	533x210x122	0.99	457x152x82	0.94
	Column	152x152x37	0.84	152x152x30	1.00
2	Brace	120x120x8 SHS	1.00	140x140x8 SHS	1.02
	Beam	610x305x179	0.85	457x191x133	0.80
	Column	203x203x46	0.93	152x152x44	0.92
1	Brace	150x150x7.1 SHS	0.94	160x160x7.1 SHS	0.98
	Beam	610x305x179	0.93	533x210x101	0.99
	Column	203x203x71	0.73	203x203x60	0.95

These differences result in smaller brace sections for the EC8 design compared to the NZS design. The beams however are typically larger for the EC8 designs due to the large in-plane point load on the beams as a result of compression braces buckling. EC8 assumes lower post-buckling brace capacity than NZS which results in a larger out-of-balance force in the braces. Table 2 summarizes the building periods, base shears and maximum permissible ductility levels for each building and design code while Table 3 presents the sections sizes for each code for the three storey building.

3.3 Initial Results of Nonlinear Time History Analysis

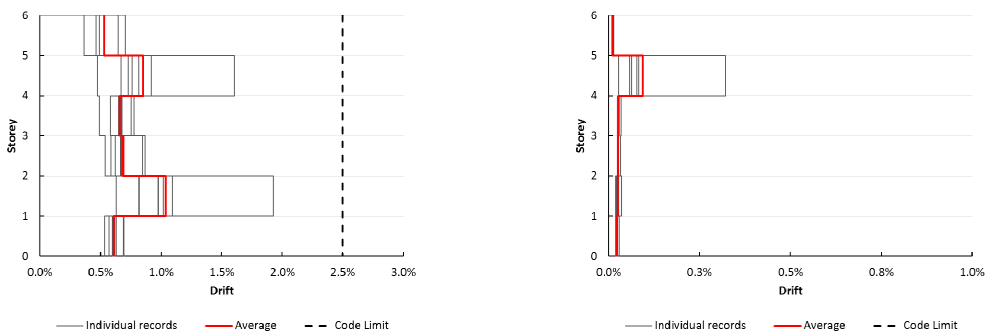
To validate the MRS results and highlighting shortcomings with the code designs, the results of the NLTH analysis are presented. For each of the NLTH analyses (EC8 and NZS), a set of seven ground motion records were selected and amplitude scaled as per EC8 scaling procedure. Each set of seven records were scaled over the period range of interest of $0.2T_1 \leq T_1 \leq 2.0T_1$, where T_1 is the fundamental period of the building in the direction under consideration. As each code design resulted in different building periods, a different scaling factor was obtained for each of the three reference buildings for both codes. The inter-storey drift results for these analyses are presented in Figure 7 to Figure 9.

The NLTH analysis was completed using SeismoStruct (SeismoSoft 2014) using force-based 3D beam-column elements with distributed plasticity and mean material properties. Connection behaviour was included using linear link elements to represent the beam-to-column shear tabs and brace gusset plates. Braces were modelled using two elements per brace. Geometric imperfections were included at the brace midpoint by defining an initial out-of-plane deformation. This is required to induce brace buckling.



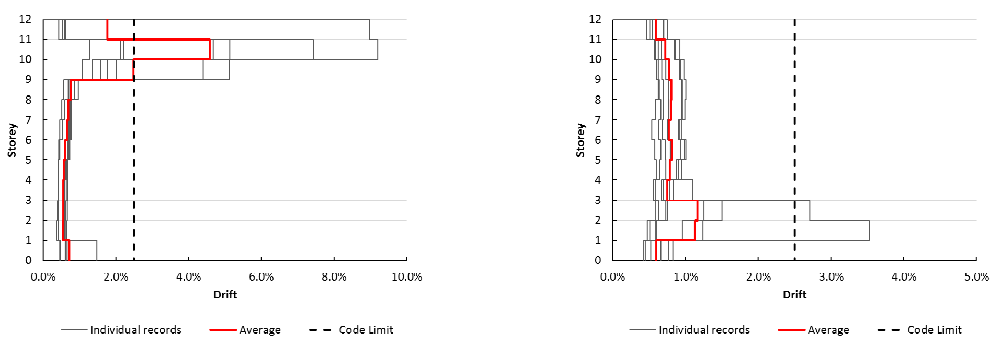
(a) (b)

Figure 7. Comparison of inter-storey drifts of NLTH analysis for (a) EC8 and (b) NZS – 3 storey building



(a) (b)

Figure 8. Comparison of inter-storey drifts of NLTH analysis for (a) EC8 and (b) NZS – 6 storey building



(a) (b)

Figure 9. Comparison of inter-storey drifts of NLTH analysis for (a) EC8 and (b) NZS – 12 storey building

As shown in Figure 7a and b, the results for the three storey building are similar with some individual records exceeding the NZS inter-storey drift limit of 2.5%. In the NZS case, the average of the seven records yield satisfactory results. With EC8, drift limitations for ULS are controlled by code prescribed ductility limitations, q, and prescriptive design checks for the type of structure in question i.e. MRF, CBF, EBF etc. In other words, the inter-storey drift limit of 2.5% is not prescribed in EC8 but is used here as a benchmark to compare results. For the three storey building, with regular vertical distribution of mass, no higher mode behaviour is apparent and most of the inelastic response is concentrated on the lower level. Of the two designs, it is the NZS design that gives the larger average inter-storey drifts.

This is interesting, as although the average response is acceptable, the more conservative design gives slightly worse behaviour.

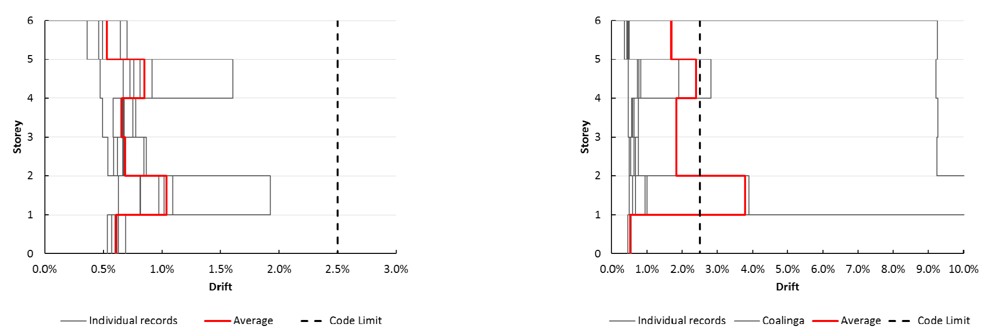
Figure 8 presents the results of the six storey building. Referring back to the code designs, EC8 utilised a ductile design while NZS was limited to a nominal ductile (near elastic) design. For both designs satisfactory behaviour is apparent, with the EC8 design showing the larger inter-storey drift by some margin. In fact, in examining the response of the steel braces for the NZS design, only the level 5 braces experienced any real inelastic response leading to the larger drift at this level. The importance of higher modes are demonstrated here.

The results of the 12 storey building are presented in Figure 9. Here, quite different responses are seen. This is somewhat expected as the NZS design (Figure 9b) requires two additional bays of bracing on levels 1, 2 and 3. Furthermore, although these levels have additional bracing, these are the levels which show the most inelastic behaviour i.e. drift. While one record for the NZS design exceeds the target, the average response is within acceptable limits and hence this design is satisfactory. In comparison, the EC8 analysis (Figure 9a) show higher modes are dominant and result in higher drifts. These exceed the NZS target drift of 2.5% by some margin at the upper most levels. However, this does not mean that the EC8 results are unacceptable. As discussed above, EC8 has no ULS drift limitation. Generally, element performance limitations (i.e. member rotations, axial strain etc) should be determined from the NLTH analysis and checked against codified element performance limits.

3.4 Results Summary

Section 3 presented the results of an elastic MRS analysis, in line with EC8 and NZS codes for three reference buildings respectively. Following this, a NLTH analysis was conducted to verify these elastic design methods. Generally speaking, acceptable results were obtained for each code with the exception of the 12 storey structure designed using EC8. The significance of higher mode effects is demonstrated in the response of these buildings under nonlinear seismic analysis.

Regarding NLTH analysis for braced frame type structures, it is worth mentioning the potential issues regarding scaling using the elastic period with all braces included in the eigenvalue analysis. The actual inelastic period of the structure is closer to the assumption that the compression braces buckle and hence don't contribute to the lateral stiffness calculation. When scaling earthquake records for NLTH analysis for braced structures, the analysist should consider these effects and check that code prescribed scaling ranges encapsulate the actual period range of interest when compression braces buckle.



(a) (b)

Figure 10. EC8 drifts from NLTH analysis for 6 storey building with (a) no degradation and (b) degradation

Element (brace) degradation, due to low cycle fatigue, is not explicitly modelled in the above NLTH analysis. However brace buckling is incorporated. Degradation due to fatigue on the brace elements under cyclic loading would decrease the performance of the structures presented in this paper and lead to increased storey displacements and likely soft storey failures. To demonstrate this point, Figure 10 presents the NLTH analysis results which include degradation of the brace elements for the six storey building for the EC8 design. As shown, once degradation is included in the analysis, the average drift increases from approximately 1% to nearly 4%. Drifts are also compared to the NZS drift limit of 2.5%.

Comparing to axial deformation criteria, as defined in Eurocode 8:Part 3, would show that member limits have been exceeded by a notable margin and the building no longer satisfies code requirements. Due to a soft storey failure, the drifts for one of the ground motions records was significant. This result is not completely shown above as it would only limit the presentation of the remaining results. Full details of the NLTH analysis including degrading properties will be covered in a future report.

4. CONCLUSIONS

This paper compares two different design procedures for the design of Inverted-V concentrically braced frames, namely Eurocode 8 (CEN 2005) and New Zealand Standard NZS 3404:Part 1 (1997). As demonstrated above, the NZS approach tends to be more conservative compared to EC8 especially for structures of six or more storeys. As explained this is primarily attributed to the larger demands imposed on the structure due to the NZS C_s factor.

This work demonstrates that there is still some potential uncertainties regarding the performance of such braced frame systems. In agreement with section 6.7.2(3) of EC8, nonlinear analysis methods should be employed to verify the behaviour of these complex systems which should include the effects of brace degradation (due to both brace buckling and low cycle fatigue).

This paper is part of a larger study into the performance of Iv-CBFs. The complete performance of all three reference buildings (included brace degradation due to low cycle fatigue) has not been explicitly presented here. This work is ongoing and final results and recommendations for refinements to the design of such frames will be included in a future publication.

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6. REFERENCES

CEN (2004). Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings. European Standard, CEN (2004). ENV 1998-1:2004.

CEN (2005). Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings. ENV 1993-1-1:2005.

Goggins, J. M., B. M. Broderick, A. Y. Elghazouli and A. S. Lucas (2005b). "Experimental cyclic response of cold-formed hollow steel bracing members." *Engineering Structures* 27(7): 977-989.

NZS 1170.5 (2004). Structural Design Actions, Part 5: Earthquake actions - New Zealand.

NZS 3404:Part 1 (1997). Steel Structures Standard.

NZS 3404:Part 2 (1997). Commentary to the Steel Structures Standard.

Remennikou, A. and G. C. Clifton (1997). Revised Expressions for Determining the Post-Buckling Compression Capacity of Category 1, 2 and 3 CBF Braces. HERA Steel Design and Construction Bulletin, No. 29.

SeismoSoft (2014). SeismoStruct v7.0 - A computer program for static and dynamic nonlinear analysis of framed structures.

Tremblay, R. (2002). "Inelastic seismic response of steel bracing members." *Journal of Constructional Steel Research* 58(5-8): 665-701.

Wijesundara, K. K., D. Bolognini, R. Nascimbene and G. M. Calvi (2009). "Review of Design Parameters of Concentrically Braced Frames with RHS Shape Braces." *Journal of Earthquake Engineering* 13(1 supp 1): 109 - 131.