
Unresolved Design Issues in Steel Eccentrically Braced Frame System

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ABSTRACT

Eccentrically braced steel frames (EBFs) are widely used in New Zealand for seismic resistance due to their ability to combine high lateral stiffness with ductile energy dissipation through yielding shear links. Current design practice, informed by early experimental research, typically relies on elastic analysis procedures as prescribed in NZS 3404 for member proportioning. However, this approach may result in non-uniform distribution of inelastic demands along the building height, with yielding potentially concentrating within a limited number of storeys and leading to excessive local deformations. Recent studies suggest that the elastic analysis methods commonly adopted in New Zealand practice may be non-conservative in predicting deformation demands. Furthermore, NZS 3404 provides limited guidance for estimating link and system-level deformation demands and adopts an elastic-perfectly plastic idealisation of link behaviour, disregarding post-yield and base rotational stiffness effects. These limitations may lead to underestimation of deformation and strength demands and introduce uncertainty in the seismic performance assessment of EBF systems.

To evaluate the accuracy of code-based estimates of shear-link rotation demands, an eight-storey EBF case-study structure designed by HERA (Clifton and Cowie, 2013) to NZS 1170.5:2004 was analysed. Seismic performance was assessed using nonlinear pushover and incremental dynamic time-history analyses, with particular focus on the effects of base rotational stiffness, link post-yield stiffness.

The results suggest that NZS 1170.5:2004 may underestimate seismic deformation demands in EBFs, especially link rotation demands, while base restraint and strain-hardening are shown to play a significant role in system response.

1 INTRODUCTION

Eccentrically braced frames (EBFs) successfully combine the advantages of the moment frames and concentrically braced frames, namely high ductility and lateral stiffness, while eliminating the shortcomings

of those frames by limiting the inelastic activity to ductile shear links and keeping braces essentially elastic without buckling, thus maintaining high lateral stiffness during earthquake events.

EBFs are characterized by an isolated segment of beam, which is referred to as link. The diagonal brace, at least at one end, is connected to the end of the link rather than the beam-column joint. All inelastic activity is intended to be confined to the properly detailed links. Links act as structural fuses which can dissipate seismic input energy without degradation of strength and stiffness, thereby limiting the forces transferred to the adjacent columns, braces, and beam segments.

Despite a substantial body of experimental and analytical research on the seismic performance of EBFs (Roeder and Popov, 1977; Kasai and Popov 1986a, 1986b; Engelhardt and Popov 1989; McDaniel, Uang, and Seible 2003; Okazaki and Engelhardt 2007; ; Mansour, Christopoulos, and Tremblay 2011, Sullivan 2013; Yu, Huang, and Sullivan 2020), further investigation remains necessary to address unresolved issues in design methodology and to enhance predictive accuracy of EBF response subjected to the seismic events. This should include the contribution of continuous columns in the gravity system of buildings which play a significant role in reducing the variation in inelastic demand up the structure.

The existing New Zealand design provisions (HERA report P4001, 2013; NZS 3404, 1997) do not adequately capture the seismic behaviour of EBFs, relying either on simplified assumptions or lacking clear guidance on critical parameters such as base rotational and post-yield stiffness. This creates uncertainty in the prediction of displacement and shear link inelastic rotation demands, particularly for taller frames where higher-mode effects and system-level interactions become more pronounced.

Accordingly, this paper aims to evaluate the seismic performance of multi-storey EBFs and to assess the reliability of the current New Zealand Steel Structures provisions (HERA report P4001, 2013; NZS 3404 1997). The study focuses on pushover capacity curve and link rotation demands through nonlinear pushover and incremental dynamic time history analyses and of an eight-storey EBF, incorporating parametric variations in base rotational stiffness and post-yield stiffness.

2 CASE STUDY

A key objective of this study is to examine possible shortcomings in the current New Zealand seismic loading standard, NZS 1170.5:2004, with respect to its ability to accurately predict deformation demands in Eccentrically Braced Frame (EBF) systems under earthquake excitation. To assess the adequacy of the code-based predictions of shear-link rotation demands, an eight-storey EBF building was selected as a representative case study. The structure was originally developed by HERA (Clifton and Cowie, 2013) and designed in accordance with the requirements of NZS 1170.5:2004.

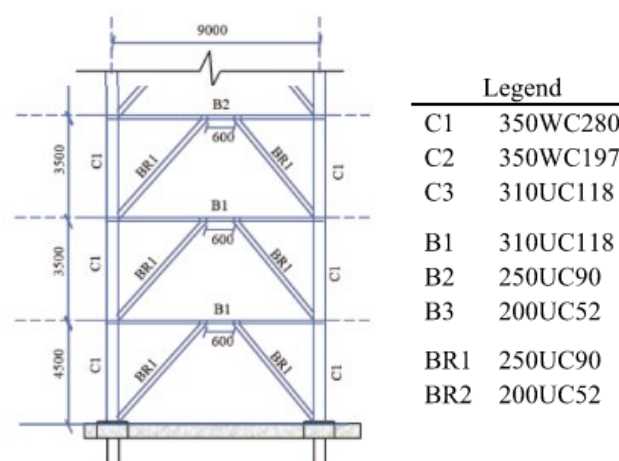


Figure 1: Case study building adopted from HERA report P4001 (2013).

3 METHODOLOGY

3.1 Current code method

Current seismic design provisions for EBFs with centrally located links are largely strength-based and follow a three-step procedure. Design base shear and ductility demands are first established in accordance with NZS 3404:1997 and NZS 1170.5:2004, based on site and soil characteristics. Lateral seismic forces are then distributed over the building height using the Equivalent Static Method, and the structural response is evaluated to obtain the predicted roof displacement, Δ_{top} .

HERA guidelines recommend that deformation demands, quantified by the inelastic rotation of shear links at each storey, be evaluated following strength design. The global drift demand of the structure is initially calculated using Eq. (1) where θ_p is the inelastic storey rotation angle; k_d is the deflection scale factor; and H is the total height of the structure. The HERA guidelines assumed that all stories have the same amount of drift demand. The inelastic link rotation, γ_p , is then calculated based on geometrical relationships, as shown in Eq. 2 where L is the span; and e is the length of the link.

$$\theta_p = \tan^{-1} \left(\frac{(\mu-1)k_d\Delta}{H} \right) \quad (1)$$

$$\gamma_p = \frac{L}{e} \theta_p \quad (2)$$

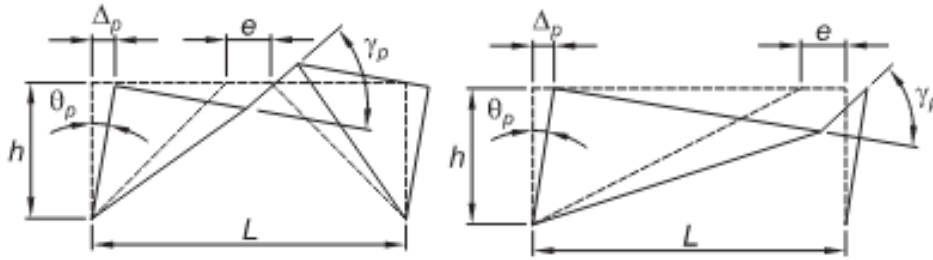


Figure 2: Link rotation angles (AISC 341-22, 2022)

The active link rotation calculated is assessed against the total deformation limit of 0.08 rad for shear-controlled links, as suggested in the current code NZS 3404 (NZS 3404, 1997).

3.2 Nonlinear Static Pushover Analysis

The seismic lateral response of an eight-storey EBF was evaluated using two-dimensional ETABS models and nonlinear pushover analyses with a triangular load pattern in accordance with NZS 1170.5:2004. Displacement-controlled loading was applied to assess deformation capacity and shear-link rotation demands.

The backbone curve and modelling parameters of active links of EBF are adopted from ASCE 41-17 and AISC 342-22 are illustrated in Figure 3 and Table 1.

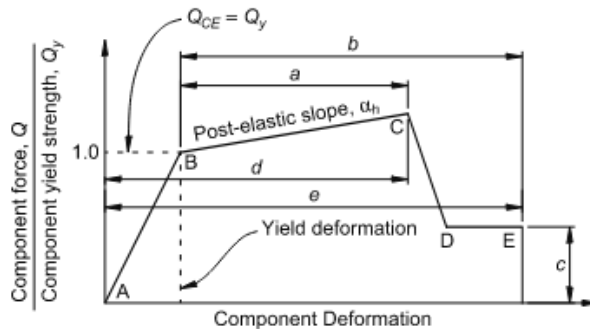


Figure 3: Generalized Force-Deformation Relation for Steel Elements (AISC342-22 & ASCE41-17)

Table 1: Modelling parameters and acceptance criteria for EBF Links (AISC 342-22 & ASCE 41-17)

Component/Action	Modelling Parameters			Acceptance Criteria		
	Plastic Rotation Angle (rad)		Residual Strength Ratio	Plastic Rotation Angle (rad)		
	a	b	c	IO	LS	CP
Shear-controlled $e \leq \frac{1.6M_{CE}}{V_{CE}}$	0.15	0.17	0.8	0.005	0.14	0.16

The yield rotation is obtained using Eq. (3) to (5), taking e (i.e., length of shear link) as L (i.e., span of the EBF). In these expressions, V_p is the corresponding plastic shear capacity, A_w is the effective shear area of the cross-section, and η represents the displacement multiplier.

$$\gamma_y = \frac{V_p}{k_e e} \quad (3)$$

$$k_e = \frac{12EI}{e^3(1+\eta)} \quad (4)$$

$$\eta = \frac{12EI}{L^2 G A_w} \quad (5)$$

For shear links, according to AISC 342-22 (2022), it is permitted to take a strain-hardening slope, a_h , for shear action as 6% of the elastic slope. In this study a strain-hardening slope of 3% of the elastic stiffness is adopted. Accordingly, the parameter r , defined as the ratio of post-yield force to yield force, is calculated as:

$$r = 1 + 0.03 (\gamma_p/\gamma_y) \quad (6)$$

The computed parameters for the shear links of the investigated EBF are summarised in Table 2.

Table 2: section properties of shear links

Shear link	$V_p(kN)$	$e(mm)$	$I(mm^4)$	$A_w(mm^2)$	η	$K_e(MN/m)$	γ_y	$r(\theta = 0.08)$	$r(\theta = 0.14)$
310UC118	675	600	2.77E+08	3780	6.35	418.51	0.0027	1.89	2.56
250UC90	524	600	1.43E+08	2860	4.34	297.80	0.0029	1.82	2.43
200UC52	316	600	5.28E+07	1648	2.78	155.28	0.0034	1.71	2.24

3.3 Incremental Dynamic Nonlinear Time History Analysis

Incremental dynamic Nonlinear direct integration time history analyses with different intensity levels were performed on the case study building with plastic hinges of shear links.

To match the seismic weight used in the HERA Report P4001 (2013), a nominal axial load of 100kN has been applied to each column to represent dead and live loads, giving a total seismic weight of 1600kN. Based on this weight, the fundamental translational period of the building, T_1 , is calculated as 0.38s.

To replicate the lateral loading from HERA Report P4001 (2013), a seismic coefficient of 2.35 is used. With a ductility factor, μ , of 4 and structural performance factor, S_p , of 0.7, the corresponding k_μ value is 2.62. The resulting base shear is $V=1000kN$ ($=1600 \times 2.35 \times 0.7/2.62$). The above acceleration is equivalent to the elastic site spectrum using the following parameters: Soil Class D, $Z = 0.4$, $R = 1.96$, and $S_p = 1.0$.

Based on the considered this elastic site spectra, seven suites of ground motions have been selected for analysis of the structure as shown in Table 3 following method A of SESOC NLRHA guideline (2025). All earthquake records are scaled to the period range of $0.4T_1$ to $1.5T_1$. Rayleigh damping with proportional damping of 5% is assigned to first and third modes of the structure.

Table 3. Input earthquakes

RSN	Event	Station	R_{rup} (km)	Mw	Scale factor (MCE)
6	Imperial Valley-02	El Centro Array #9	6.09	6.95	3.8
30	Parkfield	Shandon Array #5	9.58	6.19	3.44
33	Parkfield	Temblor pre-1969	15.96	6.19	4.24
57	San Fernando	Castaic - Old Ridge Route	22.63	6.61	4.4
77	San Fernando	Pacoima Dam (upper left abut)	1.81	6.61	1.33
95	Managua, Nicaragua-01	Managua, ESSO	4.06	6.24	2.43
96	Managua, Nicaragua-02	Managua, ESSO	4.98	5.2	5.1

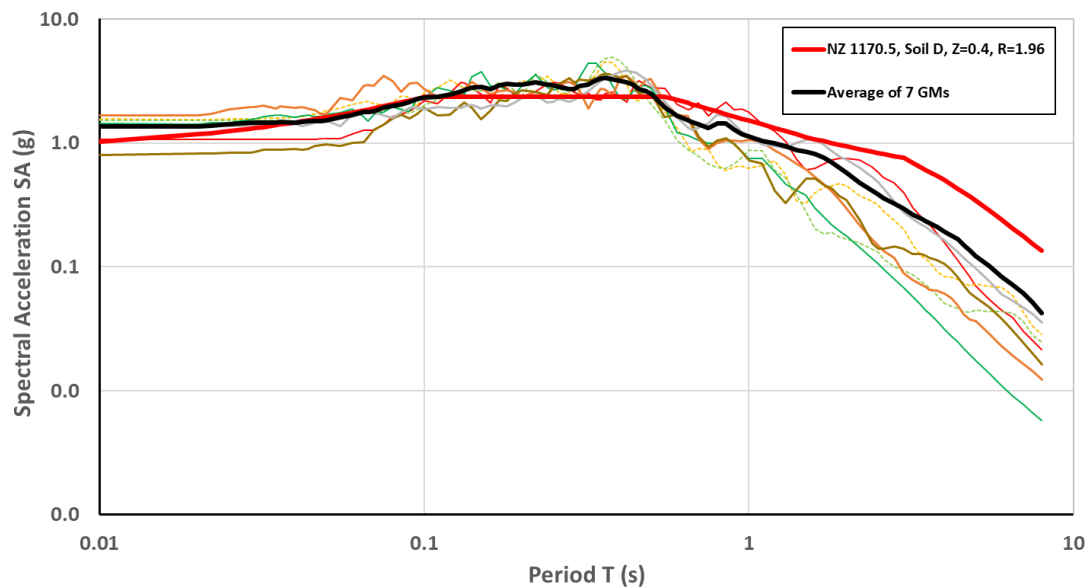


Figure 4: Spectral Acceleration of Ground Motions vs NZ1170.5 (Soil D, Z=0.4, R=1.96, $S_p=1.0$)

4 RESULT OF ANALYSIS

Figure 5 compares the peak base shear versus peak roof displacement response of the investigated EBF system with and without strain hardening. For all analyses, the expected roof displacement at the Ultimate Limit State (ULS) is approximately 180 mm. This aligns with the displacement assumed in the HERA study, where the maximum lateral roof displacement under a ductility factor of μ of 4 is 46 mm, resulting in an equivalent elastic ultimate displacement of 184 mm ($\mu \times$ displacement).

When strain hardening is ignored, the system exhibits an almost perfectly plastic response after yielding. In contrast, incorporating strain hardening results in a continued increase in base shear capacity. The results highlight that neglecting strain hardening may lead to underestimation of both lateral strength and energy dissipation capacity.

The figure also illustrates the influence of base rotational fixity on seismic response. In the elastic range, the pushover curves are closely aligned, suggesting that initial stiffness and strength are primarily governed by link behaviour rather than boundary conditions. Following yielding, however, the responses diverge, with increased base fixity enhancing strength, displacement capacity, and ductility. These effects are more pronounced when strain hardening is ignored, as pinned bases may promote excessive lower-storey yielding and weak-storey mechanisms, thereby reducing overall structural capacity; consideration of strain hardening partially mitigates these adverse effects.

Figure 5 further presents the base-shear response obtained from incremental dynamic analysis (IDA). The IDA curve represents peak responses from nonlinear time-history analyses at increasing ground-motion intensities and is constructed as the average envelope of seven scaled ground motions rather than a single record. As a result, base shear continues to increase after yielding even when zero post-elastic stiffness is assumed, primarily due to increasing inertial forces at higher intensity levels. Additional contributions arise from higher-mode effects that become more significant as the structure yields and its dynamic properties evolve, as well as from cyclic hysteretic response under load reversals, whose peak envelopes do not necessarily follow the monotonic pushover backbone.

Figure 5 also shows that, for the same base shear, lateral displacements obtained from IDA are smaller than those predicted by nonlinear static analysis, indicating that pushover analysis may underestimate displacement and plastic rotation demands. This observation is further confirmed in Figure 6, where plastic link rotations are higher under the time-history analysis across the height of the building. At the first floor of the pinned-base model, the nonlinear static analysis shows excessive yielding and a higher rotation demand compared with the time-history analysis. This difference is attributed to the averaged results of the seven ground motions and the complex higher-mode dynamic effects.

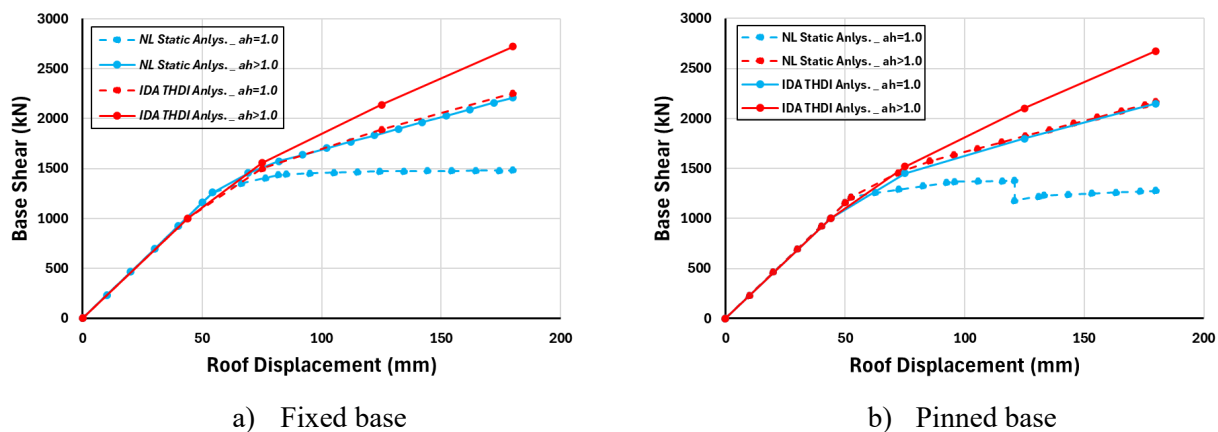


Figure 5: Peak Base shear vs. peak roof displacement with and without strain hardening.

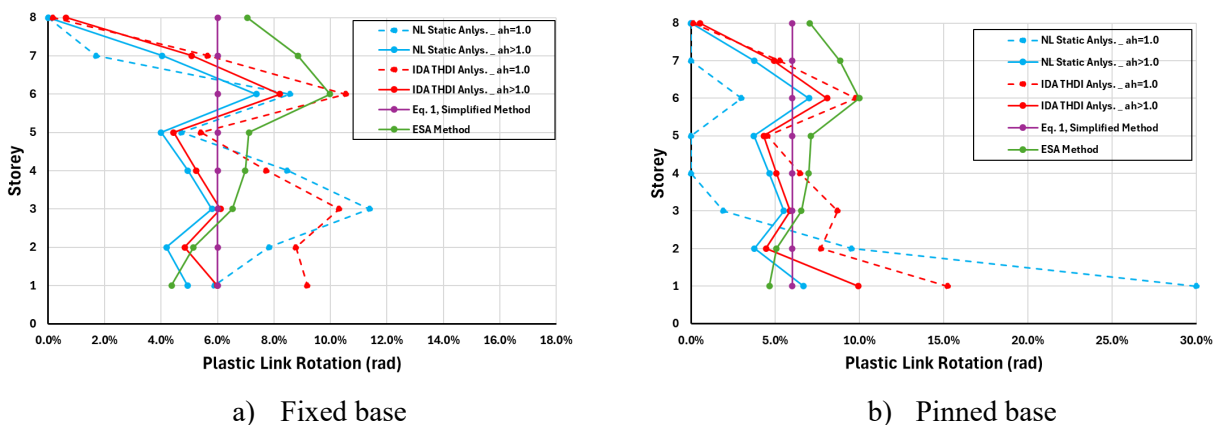


Figure 6: Plastic link rotation demand ($\Delta_{roof} = 180\text{mm}$)

Figure 6 also compares the link rotations obtained using the HERA P4001 design guidelines, as calculated from Eqs. (1) and (2) under two alternative interpretations. A common simplifying assumption in practice is to design the EBF system based on the maximum expected deformation (i.e., the roof-level displacement), which leads to the application of uniform deformation demands across all links (θ_p and γ_p), regardless of storey level. In the second and more rigorous approach, the relative displacement of each individual storey was used to compute the corresponding storey and link rotations based on equivalent static analysis (ESA).

Figure 6 highlights a clear distinction between the simplified uniform-rotation assumption and the other analytical methods (ESA). The uniform-deformation assumption overlooks the variation of inelastic demand along the height of the structure and can therefore either overestimate or underestimate local link rotations, depending on the storey. In contrast, when considering the post-elastic stiffness (which reflects the actual structural behaviour and the influence of the floor slab), the ESA method fails to capture the plastic rotations at the lower levels and predicts significantly higher rotation demands at the upper levels compared with the nonlinear analysis.

Overall, the comparison demonstrates the limitations of both the simplified uniform-rotation assumption and the ESA method: the first lacks the resolution to represent the true variability of link rotations along the height, while the second does not reflect actual structural behaviour and is overly conservative.

5 CONCLUSION

Based on the non-linear static pushover analysis and incremental dynamic Nonlinear analyses conducted on the prototype buildings using the real ground motions, the following conclusions can be made:

1. Neglecting strain hardening produces an elastic–perfectly-plastic response with earlier strength plateau and concentrated plasticity in the lower storeys, particularly at the first level. Incorporating post-yield stiffness (per ASCE 41-17) increases base-shear capacity, redistributes plastic rotations over multiple storeys, reduces peak link demands, and improves overall ductility.
2. Increasing rotational restraint at the base connections from pinned to partial/near fixed enhances lateral strength retention and extends displacement capacity. Higher fixity reduces concentration of plastic rotations in lower storeys and promotes a more uniform spread of inelasticity, thereby increasing system ductility.
3. Current New Zealand standard and design guidelines may underestimate deformation and plastic link rotation demands in EBF structures. The results of this study highlight the limitations associated with both the simplified uniform link rotation method and ESA method.

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