Performance-based seismic design of intentionally eccentric IH-treated steel braced frames

Whittall, Thomas; Skalomenos, Konstantinos

Citation for published version (Harvard):
Whittall, T & Skalomenos, K 2022, Performance-based seismic design of intentionally eccentric IH-treated steel braced frames. in *ICONHIC2022 Proceedings .*, 479, International Conference on Natural Hazards and Infrastructure, National Technical University of Athens, Athens, Greece, 3rd International Conference on Natural Hazards and Infrastructure, ICONHIC 2022, Athens, Greece, 5/07/22.

ICONHIC2022
3rd International Conference on Natural Hazards & Infrastructure
5-7 July 2022, Athens, Greece

Performance-based seismic design of intentionally eccentric IH-treated steel braced frames

Thomas Whittall, MEng, and Konstantinos Skalomenos, PhD
Department of Civil Engineering, Edgbaston, Birmingham, B15 2TT, University of Birmingham

ABSTRACT

Using induction heat (IH) treatment to increase the yield stress of one half of a steel brace section (i.e., a dual-strength steel section), as well as inducing intentional eccentricity along the brace length has been experimentally proven to increase the limited post-yielding stiffness exhibited by concentrically braced steel structures. This paper aims to numerically investigate the seismic performance of steel braced frames using the IH-treated steel sections with intentional eccentricity and establish a performance-based seismic design method for them. More specifically, a physical model is developed to calculate the multiple strength points, as well as the increased post-yielding stiffness of the brace. On the basis of the physical model, mathematical expressions are developed to support the seismic design of the proposed braced frame structures. The high post-yielding stiffness and controllability of the brace response through the effective combination of the IH-treated steel section and eccentricity provide the brace the capability of satisfying multiple strengths and deformation performance objectives offering reduced section sizes. Time-history analysis results under three hazard levels (frequent, design-basis and maximum occurring event) demonstrated that better control is achieved with the proposed bracing system in achieving drift and ductility limitations dissipating more evenly the seismic energy along the height of the structure. A significant reduction of the residual deformation without storey damage concentration was observed at high seismic intensity levels.

Keywords: Multi-objective design, Steel braced structures, Induction heating, Post-yielding stiffness

INTRODUCTION

A new focus on reducing post-hazard financial losses and recovery time, alongside advances in computational methods, has culminated in the emergence of Performance Based Seismic Design (PBSD). Within PBSD, buildings are designed to respond to seismic loading predictably and reliably to various levels of structural performance, increasing hazard resilience. However, the complex inelastic behaviour of certain conventional structure types, such as concentric braced frames, provides difficulties in the reliable prediction of the non-linear response for these structure types. New systems are under development with controlled inelastic behaviour, providing satisfaction of PBSD to multi-level objectives [1-3].

Concentric Braced Frames (CBFs) are a prevalent type of steel braced frame system that is characterised by the use of diagonal bracing members that join with the endpoints of beams and columns, forming a vertical truss. Lateral seismic loads are transmitted through axial tension and compression forces in the bracing members. Bracing members in CBFs also act as dissipative elements through yielding in tension and buckling and post-buckling deformation in compression [4]. Compared to other lateral force resisting systems, CBFs generally provide large levels of stiffness and lateral strength but possess limited ductility capacity under cyclic loading [5]. CBFs exhibit an hysteretic response to seismic loading that make brace members susceptible to early buckling failures and a low post-yielding stiffness [6]. The poor inelastic behaviour of CBFs has provided difficulties in the application of PBSD and reliable prediction of their response to seismic loading.

In consideration of simple design and improving inelastic brace behavior, a brace with intentional eccentricity (BIE) has been recently proposed [7]. An eccentricity is induced along the length of the member and under axial loading undergoes overall bending, reducing stress concentrations and delaying local buckling. A positive post-yielding is produced in the brace with a tri-linear backbone curve in tensile loading. Skalomenos et al. [8, 9] investigated applying induction-heating (IH) to one half of the BIEs, giving a two-component design, which greatly increases post-yielding stiffness and yield strength, but decreases fracture ductility. The inelastic behavior and design quantities of induction heated braces with intentional eccentricity (IH-BIEs) have been experimentally developed and validated, allowing the formulation of a design methodology. A steel braced frame that uses IH-BIEs are capable of a multi-level seismic design so that one design can satisfy multiple strength objectives. The purpose of this paper is to present a new multi-level hazardous design methodology for steel braced frames incorporating the two-component IH-treated BIEs, within the framework of PBSD.

METHODOLOGY

Brace design and mechanical behaviour

Figure 1a shows the design of a single-phase brace with intentional eccentricity and a two-phase induction heated brace with intentional eccentricity (IH-BIE). A conventional hollow steel section is arranged with an eccentricity from the normal working points of the brace, and for an IH-BIE, is induction heated over one half of the section. The axial loads are transferred to the frame through rigid end-to-brace gusset plate connections. Figure 1b shows the force-deformation behavior of both a BIE and an IH-BIE in tension and compression, compared to a conventional brace. Contrary to the bilinear response of the conventional brace, BIEs exhibit a trilinear backbone curve in tension, with a reduced elastic stiffness and yield point. The brace possesses greater post-yielding stiffness, increasing from the first yielding point to a final value governed by axial stiffness. In the IH-BIE, steel is heated and quenched to give one half of the section properties similar to high strength steel. As shown in Fig. 1b, the result is an improved ultimate tensile strength, yield strength, compressive strength, and a much larger post-yielding stiffness compared to a BIE.

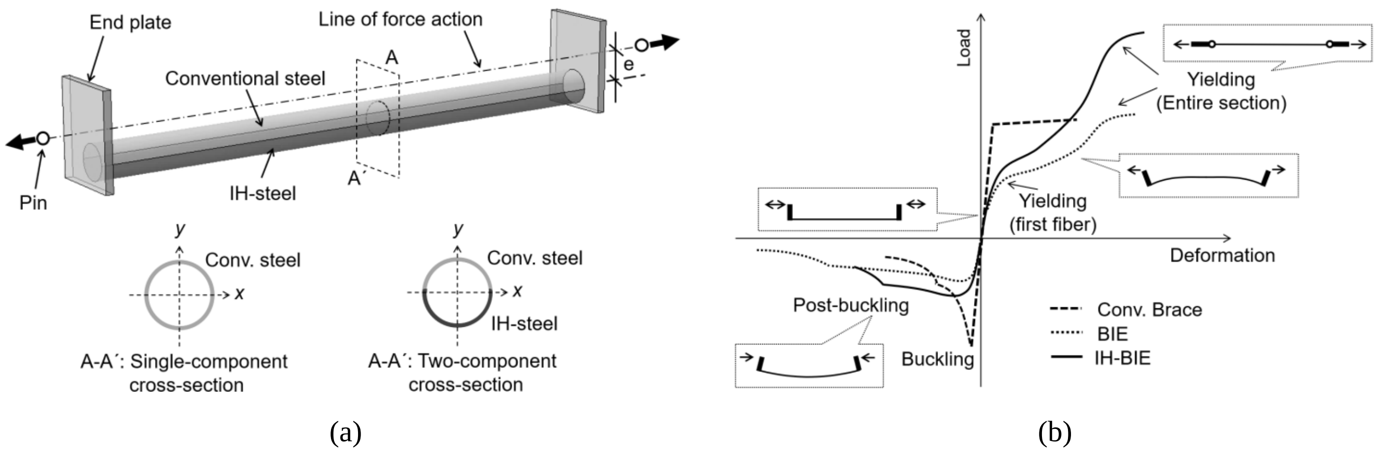


Figure 1. *(a) 3D model of braces with intentional eccentricity with a single or a two-component cross-section; (b) comparison of backbone curves of conventional steel braces, BIEs and IH-BIEs based on test findings*

For a multi-level objective design, that is when more than one level of structural performance is specified, the zero or negative post-yielding stiffness in conventional braces may lead to the design of uneconomical structures [10, 11, 12]. The design is dominated by the more strenuous objective, leading to an increase in strength demand and larger brace sections. Figure 2 shows how positive post-yielding systems produce a trilinear load-displacement curve, as opposed to conventional braces, satisfying multi-level objective design more rationally.

In the proposed intentionally eccentric IH-treated steel braced frames (IH-FIEB), a ‘first yield’ point corresponding to full yielding of the conventional steel of a single brace is denoted by P_{y1} and a ‘second yield’ point corresponding to full yielding of the IH-treated steel of a single brace is denoted by P_{y2}. The brace is assumed to act elastic-plastically in compression, and the compressive strength, P_c, is taken as the same as the first yield point, $P_{y1} = P_c$. Figure 2 shows the trilinear curve of a chevron IH-FIEB (“c” indicates chevron braces). In the proposed force-based design of IH-FIEB, it is assumed that the conventional steel half-section yields under the frequent seismic event, while the IH-steel half-section yields under the major seismic event.

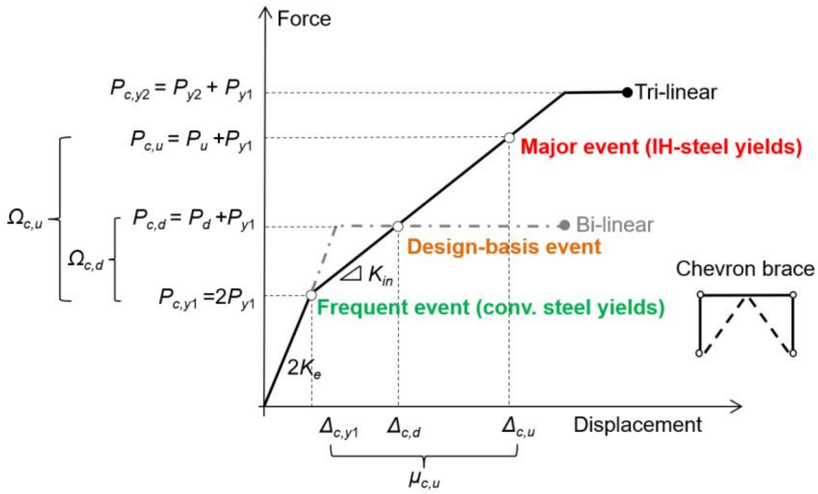


Figure 2. *Performance based seismic design of IH-FIEB.*

Design equations

IH-BIEs can be designed by controlling both the IH-ratio and eccentricity, where the IH-ratio is defined as the ratio of the yield strength of the induction heated treated steel to the yield strength of the conventional steel. By varying both IH-ratio and eccentricity, controllability of the yield points and post-yielding stiffness is afforded, allowing IH-BIEs to be adjustable to satisfy required performance objectives. This section introduces relationships that describe the overall response of the brace and how these are used to formulate a multi-level design method. Equations are developed mainly through the strength ratio, Ω, as shown in Fig. 2. By employing the set of equations, someone can design the brace to satisfy the required performance objectives. In this design method, three hazard levels of seismic design are considered that correspond to a frequent, design-basis and major event, i.e., immediate occupancy (IO), life safety (LS) and collapse (C), respectively.

Skalomenos et al. [8] found that the IH steel has a fracture ductility 3 times lower than conventional steel and it is possible fracture of the IH-steel occurs before P_{y2} is reached. In design, it is therefore proposed that a safety factor is applied to give a lower ultimate tensile strength, P_u, preventing premature fracture of the brace. A value of γ_u equal to 0.6 was found sufficient to limit the IH-steel from yielding for an IH-ratio equal to 4. In this case, γ_u and γ_d are values that theoretically relate to the proportion of the brace section that is expected to remain elastic in the major event and the design-basis event respectively, where $\gamma = 1$ represents a completely plastic section. Therefore, γ_u can be used in the design process to limit the yielding of the IH-steel. More discussion about this can be found elsewhere [13].

Considering a brace pair in tension and compression, the strength ratio Ω can be defined for each seismic hazard level as the ratio of the strength at the target level to the strength at the first yield point, P_{y1}. For the major event the ultimate strength ratio is symbolized as $\Omega_{c,u}$ while for the design-basis event the design strength ratio is symbolized as $\Omega_{c,d}$. Equation 1 gives the relationship in strength for $\Omega_{c,u}$ and Equation 2 gives the relationship in strength for $\Omega_{c,d}$, where γ_u is the safety factor limiting yielding of the IH steel, P_u is the factored ultimate tensile strength of a single brace and P_d is the tensile strength of a single brace designed for the design-basis event. The value of the factored ultimate tensile strength is given in terms of γ_u, P_{y1} and P_{y2} in Equation 3.

$$\Omega_{c,u} = \frac{P_{c,u}}{P_{c,y1}} = \frac{P_u + P_{y1}}{2 \cdot P_{y1}} = 1 - \frac{\gamma_u}{2} \cdot \left(1 - \frac{P_{y2}}{P_{y1}}\right) \tag{1}$$

$$\Omega_{c,d} = \frac{P_{c,d}}{P_{c,y1}} = \frac{P_d + P_{y1}}{2 \cdot P_{y1}} = 1 - \frac{\gamma_d}{2} \cdot \left(1 - \frac{P_{y2}}{P_{y1}}\right) \tag{2}$$

$$P_u = (1 - \gamma_u)P_{y1} + \gamma_u P_{y2} \tag{3}$$

The ratio P_{y2}/ P_{y1} is obtained from the defined strength ratios, which is then used to the ratio of eccentricity and radius of gyration using Equation 4, considering the level of induction heating on the section. Finally, the brace area can be calculated using Equation 5 for a given story shear, V_i, the calculated e/r ratio, the yield strength of the conventional steel and the ratio $\Omega_{c,u}$. Eccentricity can be determined from radius of gyration in the ratio e/r, after brace area has been selected, giving the brace the required properties to meet the strength requirement at each hazard level, and to maintain the same brace overstrength between each story.

$$\frac{P_{y2}}{P_{y1}} = \frac{1}{2} \cdot \left(1 + \sqrt{2}\frac{e}{r}\right) \cdot \left[\left(\frac{F_{u,CS}}{F_{y,CS}}\right) + \left(\frac{F_{y,IH}}{F_{y,CS}}\right)\right] \tag{4}$$

$$A_{req} = \left(1 + \sqrt{2}\frac{e}{r}\right)\frac{V_i}{2 \cdot \Omega_d} \cdot \frac{1}{F_{y,CS}} \tag{5}$$

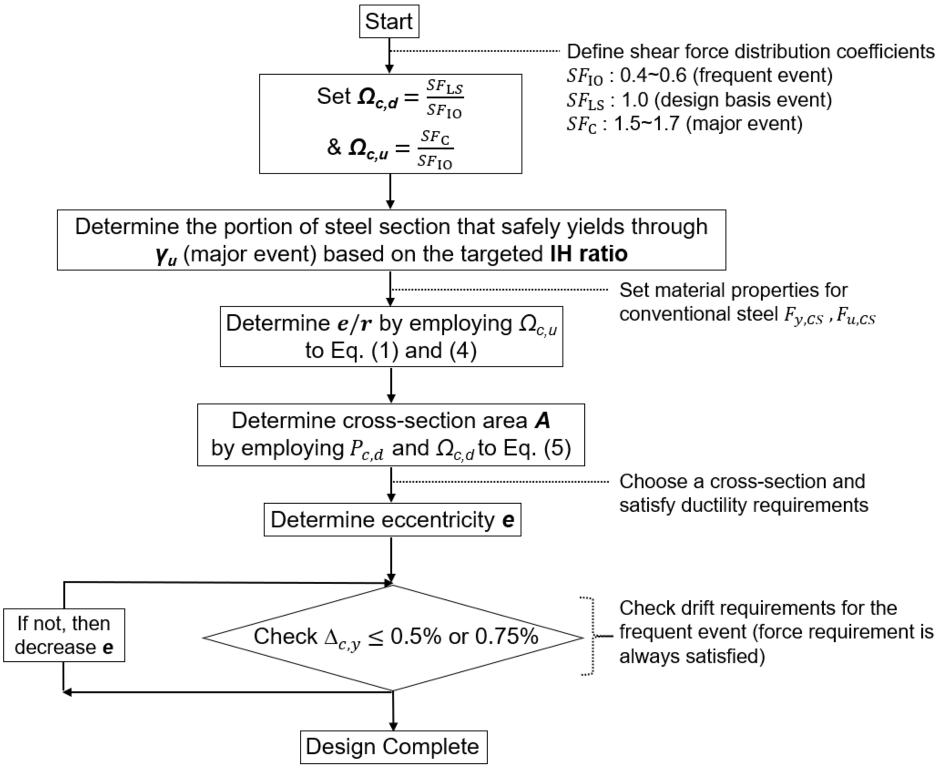


Figure 3. *Flowchart for design of IH-FIEB*

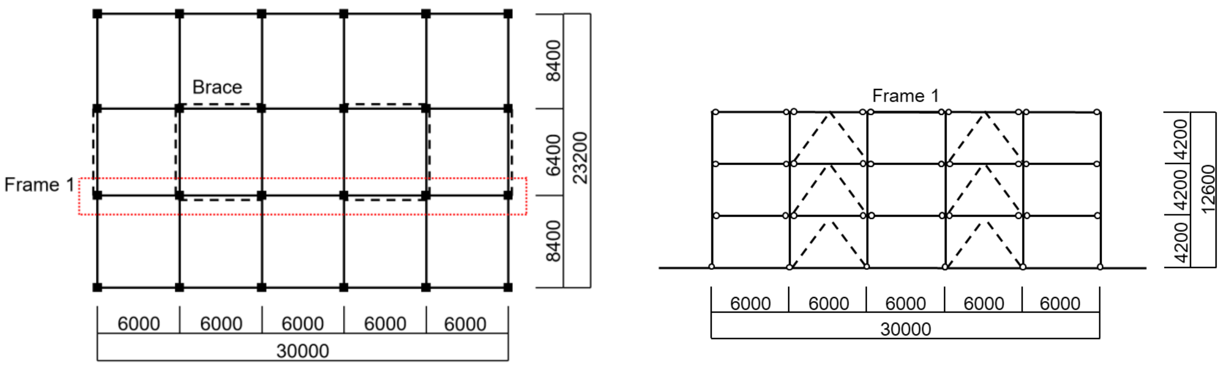
Design flowchart

This section introduces the flowchart for the design method, including defining the strength ratio for each performance level, in the context of Eurocode 8 (EC8) [14]. In the design basis event, the no-collapse requirement is the target, under the reference seismic action associated with a reference probability of exceedance of 10% in 50 years, or a reference return period 475 years. Accordingly, in the frequent event, the damage limitation requirement is the target to be met under a seismic action having a larger probability of occurrence than the design seismic action of 10% in 10 years, or a return period of 95 years. In EC8, there is no recommendation for a maximum considered earthquake, although design verifications provide implicit equivalence to satisfaction of a global collapse target under a very rare event, approximately with a 1500-to-2500-years return period. The scale factor required to multiply the design-basis spectral acceleration coordinates for the frequent and major events may be computed as $(T_{\mathrm{LR}}/ T_{\mathrm{L}})^{-1/3}$, where T_{L} = 475 years and T_{LR} is the reference return period of the frequent or major event. The ratio of these shear force distribution coefficients is then used to find the strength ratios, used in the first step of the design procedure. Based on the targeted IH ratio, the safety parameter γ_u is selected to control the portion of the cross-section that shall remain elastic in the major event, preventing yielding of the IH steel. The *e/r* ratio of the brace can now be found using Equation 1 and Equation 4 and subsequently the required brace area using Equation 5. By using the eccentricity demanded by the *e/r* ratio a design of a brace pair is produced that satisfies the required overstrength and post-yielding stiffness to meet the strength objectives. The eccentricity can be adjusted to meet the drift requirements for the frequent event, or to ensure uniform distribution of displacements.

Design example and time-history analysis

This section applies the proposed design method in a prototype office building. Two structures were designed, one with IH-BIEs (IH-FIEB) and a concentrically braced frame (CBF) designed to EC8 for comparison. The prototype building is a three-story five-span braced frame, as shown in Figure 4. The plan view of the building is shown in Fig. 4a and has 30 m width and 23.2 m depth. The braces are placed in chevron and their locations are illustrated by dashed lines in the plan view. The elevated view of the building is shown in Fig. 4b. The seismic forces are resisted only by the bracing system. The total vertical load (i.e. seismic mass) of the structure is equal to 7.8 kN/m^2 for the first floor, 7.7 kN/m^2 for the second floor, and 8.9 kN/m^2 for the third floor. The seismic base shear under the design-basis event is calculated using the design spectrum for elastic analysis of EC8 [14] with a_{gR} = 0.35g, importance factor 1.0, soil type B. Thus, the spectral acceleration at the fundamental period T_1 of the structure Sa (T_1) = 0.35g. The design base shear coefficient for the frequent event is 0.6·0.35g = 0.21g and for the major event is 1.73·0.35g = 0.595g. These values define a strength ratio $\Omega_{c,d} = 1.67$ and $\Omega_{c,u} = 2.89$.

An IH-ratio of 4 was targeted, as this provides the most economical solution, due to reduced section sizes. A higher q factor, q = 3, was adopted here for IH-FIEB than what is recommended by EC8 for braced structures to account for the higher ductility of the IH-BIEs. Moreover, yielding is allowed for the frequent event to enable an early energy dissipating behaviour. For CBFs, q factor equals to 2.5. The design results of the bracing systems are shown in Table 1. For the compression members, the buckling load was calculated. Columns and beams were designed to remain elastic in the IH-FIEB by considering the maximum design forces produced by the IH-BIEs. Braced bay beams are designed with HEB sections due to the large, unbalanced forces produced by inverted-V braces in tension and compression. For FIEBs this is taken as the vertical component force produced as a result of the difference between P_d, the expected resistance in tension and P_{y1}, the expected resistance in compression. (FIEB: 1st HEB 400, 2nd HEB 400, 3rd HEB 340 and CBF: 1st HEB 450, 2nd HEB 450, 3rd HEB 400). Columns in the FIEB are a SHS 350 × 350 × 10 section and in the conventional frame a SHS 350 × 350 × 12 section. The total steel tonnage of a single seismic resisting frame in the IH-FIEB is reduced by 8.4% compared to the CBF, with a maximum reduction of 12% in the top floor. Table 2 shows the mechanical characteristics of one chevron braced span expressed in lateral forces and storey drifts.



(a) (b)

Figure 4. *Overview of the target building (dimensions in mm): (a) plan; and (b) elevation*

Table 1. *Dimensions of steel brace sections in the seismic resisting frame*

	Storey	Section (mm)		λ	Steel (MPa)		e (mm)	IH ratio	e/r
		D	t		$F_{y,CS}$	$F_{u,CS}$			
CSB	1	323.9	8.0	41.6	235	400	-	-	-
	2	273.1	8.0	49.5	235	400	-	-	-
	3	193.7	8.0	70.7	235	400	-	-	-
IH-BIE	1	273.1	10.0	47.6	235	400	100	4.0	1.10
	2	244.5	10.0	53.1	235	400	90	4.0	1.10
	3	219.1	7.1	67.3	235	400	80	4.0	1.20

Table 2. *Mechanical characteristics of one chevron braced span expressed in lateral forces and storey drifts*

		Frequent event		Design-basis event		Major event	
	Storey	$K_{e,c}$ (kN/mm)	$P_{c,y1}$ (kN)	$K_{in,c}$ (kN/mm)	$P_{c,d}$ (kN)	$P_{c,u}$ (kN)	$P_{c,y2}$ (kN)
CBB	1	232.57	1576.0	-	1736.6	-	-
	2	194.30	1258.0	-	1434.0	-	-
	3	137.14	785.3	-	953.7	-	-
IHBIE	1	110.67	875.8	39.1	1464.0	2542.6	3653.8
	2	97.62	774.0	34.5	1299.0	2261.6	3253.3
	3	63.79	504.5	22.6	840.6	1457.0	2092.1

To perform nonlinear time-history analysis (NTHA), the finite element analysis software framework OpenSees [15] was used to develop a mixed fibre-distributed frame model of the building. The braces were modelled with the induction heating properties on one half, and with a constant eccentricity. Eight existing ground motion histories were modified to fit the targeted EC8 design elastic response spectrum. The ground motions were scaled by a factor of 0.3 and 1.73, to account for the frequent and maximum event respectively. More details about the nonlinear modelling can be found elsewhere [13].

RESULTS AND DISCUSSION

Under the design-basis and major events, ASCE 41-13 [16] restricts inelastic story drifts to 1.5% and 2.0%, respectively, while EC8 [14] gives no recommendation. Figure 5 shows the comparison between the maximum interstory drift at the frequent, design and major event in the conventional frame and the IH-FIEB. The drift limitations are indicated through vertical red dash lines in each figure. In all frequent event ground motions, the IH-FIEB and CBF do not exceed 0.5% story drift, and both experience the peak mean story drift in the top story, with values of 0.34% and 0.37%, respectively. Under the design event, the IH-FIEB experiences peak mean story drift values between 0.85% and 0.95%, whereas the CBF ranges between 0.41% and 0.85% story drift. In the major event, the CBF experiences a lower peak mean story drift in all stories compared the IH-FIEB. Across all events, the conventional frame designed in accordance with EC8 [14] experiences a cantilever response, with the maximum drift experienced in the top one or two stories. The IH-FIEB experiences a response close to a shear building, with only small differences in the maximum interstory drift between the stories. Conventional braced frames exhibit poor plastic engagement and a concentration of damage at the roof, as EC8 rules on the variation of brace overstrength ratios between each storey do not assure uniform plastic engagement. In comparison, the proposed force-based design procedure for IH-FIEBs allows a controllability of the brace through eccentricity, providing a more uniform overstrength and drift demand. Drift concentrations and the potential for a soft-storey mechanism are prevented by the combination of the design method and the ability of IH-BIEs to smoothly transition into post-buckling and exhibit a more uniform plastic engagement at a lower drift level.

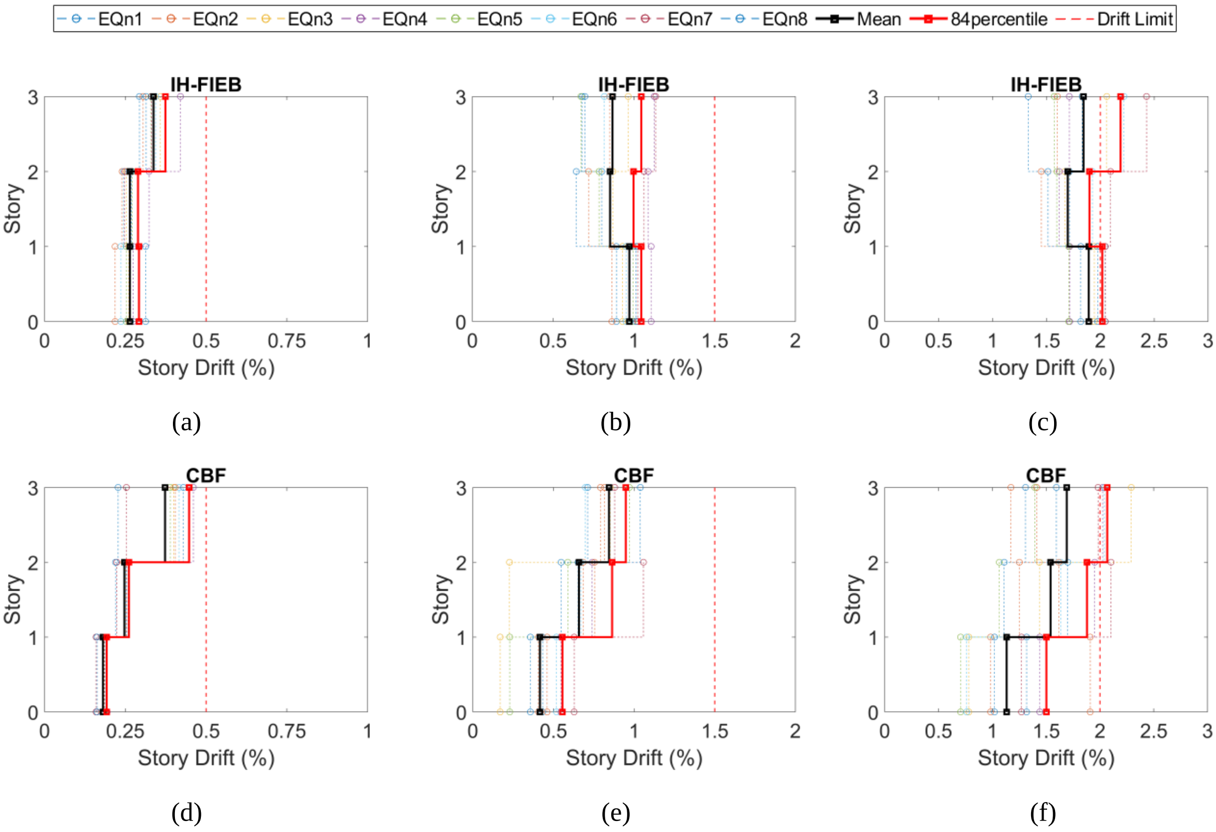


Figure 5. *Maximum story drifts for each hazard level in IH-FIEB and CBF frames: (a) IH-FIEB – frequent event; (b) IH-FIEB – design-basis event; (c) IH-FIEB – major event; (d) CBF – frequent event; (e) CBF – design-basis event; (f) CBF – major event*

Figure 6 shows the normalized force-deformation relationship for both an IH-BIE and a conventional steel brace in the 3rd story of the building under the major event. Drift is normalized to ductility and force is normalized to strength ratio in order to compare each brace. It can be seen that the IH-BIE provides a more stable, reliable, and symmetric behavior, with a large positive post-yielding stiffness. The smoother transition to buckling provides the desirable energy dissipation and plastic engagement of the IH-BIEs.

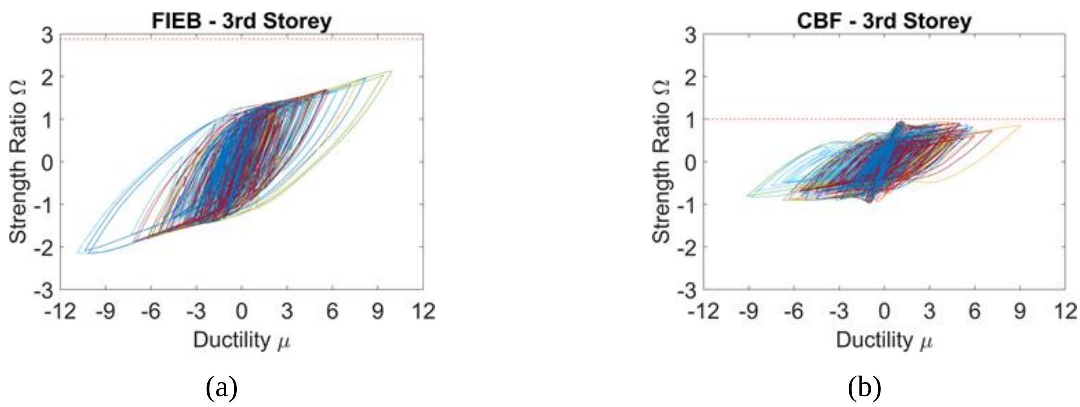
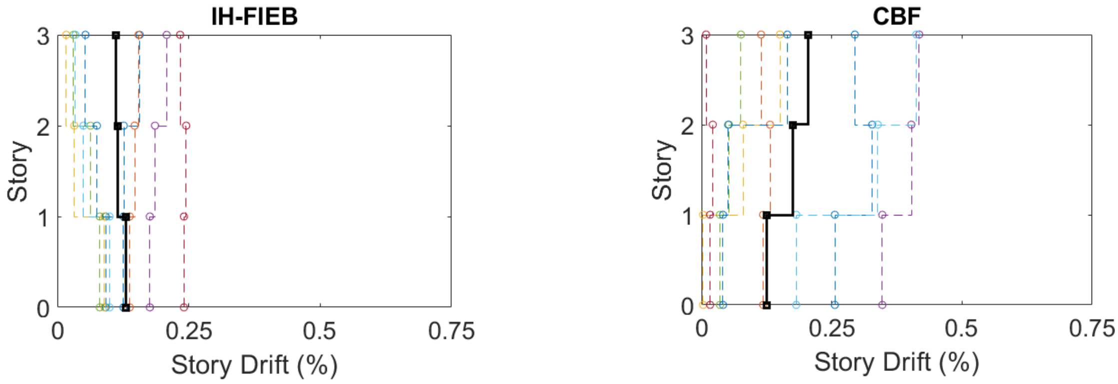


Figure 6. *Normalized lateral load – story drift relationships for the IHBIE and conventional brace for the major event: (a) IHBIE – 3rd story; (b) CBF – 3rd story*

Figure 7 shows that the IH-FIEB produces smaller residual drifts on average than the conventional frame, possibly due to the positive post-yielding stiffness preventing excessive deformation. This indicates that IH-BIEs may be able to reduce post-hazard damage and associated costs of repairability compared to CBFs.



(a) (b)

Figure 7 *Residual inter-story drifts after major event - (a) FIEB (b) CBF*

CONCLUSIONS

The main findings of this paper are as follows:

- Eccentricity and IH treatment enable a multi-level seismic design method to be developed within the framework of Performance Based Seismic Design. Proposed braced structures exhibit a multi-phase yielding with a stabilized buckling behaviour and high post-yielding stiffness, delaying local buckling.
- The proposed braced structures can meet strength criteria at different levels of seismic hazard simultaneously. The design method adequately controls the drift demand and story shear of FIEBs, providing satisfaction of all three strength performance objectives.
- The design method ensures the major event strength requirement is met by increasing the *e/r* ratio. This naturally results to higher drift demand for the IH-FIEB compared to the CBF. The eccentricity could be decreased to achieve lower drifts.
- Compared to CBF, FIEBs exhibit better plastic engagement, a more uniform drift demand and a more stable energy dissipation. IH-BIEs may be able to limit residual drifts, provide a reduction in the damage concentrations observed in CBFs and reduce total steel tonnage of a building structure adopting higher behavior factor *q* (strength reduction factor) for IH-FIEBs.
- IH-FIEB appear to produce smaller residual drifts on average than the conventional frame, possibly due to the positive post-yielding stiffness that prevents excessive deformation. After the major event, the top story of CBFs exhibited residual drifts nearly at 0.25%, while those of IH-FIEB were found to be 50% lower. This indicates that IH-BIEs may be able to reduce post-hazard damage and associated costs of repairability compared to CBFs.

REFERENCES

[1] Asgarian B., Moradi S. (2011). Seismic response of steel braced frames with shape memory alloy braces. Journal of Constructional Steel Research, 67(1), pp. 65-74.

[2] Gray M.G., Christopoulos C., Packer J.A. (2014). Cast Steel Yielding Brace System for Concentrically Braced Frames: Concept Development and Experimental Validations. Journal of Structural Engineering of ASCE, 140(4), 04013095

[3] Shen J., Seker O., Sutchiewcharn N., Akbas B. (2016). Cyclic behavior of buckling-controlled braces. Journal of Constructional Steel Research, 121, pp. 110-125.

[4] Jin J., El-Tawil S. (2003). Inelastic Cyclic Model for Steel Braces. Journal of Engineering Mechanics, 129(5), pp. 548-557.

[5] Gioncu V., Mazzolani, F. Earthquake Engineering for Structural Design. New York: Soon Press; 2011.

[6] Lai J.-W., Mahin S. A. (2014). Steel concentrically braced frames using tubular structural sections as bracing members: Design, full-scale testing and numerical simulation. International Journal of Steel Structures, 14, pp. 43-58.

[7] Skalomenos K.A., Inamasu H., Shimada H., Nakashima M. (2017). Development of a steel brace with intentional eccentricity and experimental validation. Journal of Structural Engineering of ASCE, 143(8): 04017072.

[8] Skalomenos K.A., Kurata M., Shimada H., Nishiyama M. (2018). Use of induction heating in steel structures: material properties and novel brace design. Journal of Constructional Steel Research, 148, pp. 112-123.

[9] Skalomenos K.A., Kurata M., Nishiyama M. (2020). Induction-heat treated steel braces with intentional eccentricity. Engineering Structures, 211, 110461.

[10] Ye L., Lu X., Ma Q., Cheng G., Song S., Miao, Z., Pan P (2008). Study on the influence of post-yielding stiffness to the seismic response of building structures. 14th World Conference on Earthquake Engineering. Beijing, China.

[11] Iemura H., Takahashi Y., Sogabe, N. (2006). Two-level seismic design method using post-yield stiffness and its application to unbonded bar reinforced concrete piers. Structural Engineering / Earthquake Engineering, 23(1), pp. 109s-116s.

[12] Kiggins S., Uang C.M. (2006). Reducing residual drift of buckling-restrained braced frames as a dual system. Engineering Structures, 28 (11), pp. 1525–1532.

[13] Skalomenos K., Whittall T., Kurata M., Pickering J (2022). Component testing and multi-level seismic design of steel braced frames with high post-yielding stiffness and two-phase yielding, Soil Dynamics and Earthquake Engineering, 107248 (in-press)

[14] Eurocode 8. Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings. EN 1998-1. Brussels: European Committee for Standardization; 2004.

[15] McKenna F., Fenves G.L., Scott M.H. Open System for Earthquake Engineering Simulation. Berkeley: University of California; 2000

[16] ASCE. Seismic Evaluation of Retrofit of Existing Buildings. Standard ASCE/SEI 41-17. Reston, Virginia, USA: American Society of Civil Engineers; 2017.