Soil Dynamics and Earthquake Engineering 115 (2018) 27–35

Contents lists available at ScienceDirect

Soil Dynamics and Earthquake Engineering

journal homepage: www.elsevier.com/locate/soildyn

On the seismic behaviour of tension-only concentrically braced steel structures

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ARTICLE INFO

Keywords:
Tension-only braces
Slotted holes
Seismic analysis
Three-dimensional steel structures

ABSTRACT

This paper summarizes the estimated results from three-dimensional non-linear time-history seismic analysis of tension-only concentrically braced steel structures. The braces of these type of steel structures are properly detailed in order to sustain only tension and no compression. In particular, a cheap and easy to fabricate brace detailing allows the brace to slide when in compression and to develop a resisting force when in tension. A comparison between steel structures designed with the proposed tension-only braces and with buckling-restrained braces is performed on the basis of commonly used seismic response and demand indices. It is shown that tension-only and buckling-restrained braced structures may exhibit similar behavior. Nevertheless, column overstress in compression is larger for the tension-only braced structures. Preliminary conclusions regarding the use of the proposed tension-only braces as a seismic force-resisting system for steel structures are drawn.

1. Introduction

Concentrically braced frames (CBFs) constitute a popular seismic force-resisting system for steel structures. They are typically separated in ordinary concentrically braced frames (OCBFs) and special concentrically braced frames (SCBFs). Seismic codes distinguish these two types of CBFs by enforcing appropriate design and detailing requirements, even though OCBFs generally are not recommended for areas of high seismicity [1,2]. Useful overviews on the seismic behaviour of CBFs and of SCBFs taking into account the properties and the configurations of the braces can be found in literature, e.g., [1–4] and references therein.

Buckling-restrained braced frames (BRBFs) are a special type of CBFs where braces are appropriately detailed against global buckling and strength loss [1,2,4,5]. A BRBF is usually more flexible than a SCBF and its design is governed by code-specified drift limits [1]. BRBFs tend to concentrate damage in specific storeys producing large permanent drifts [1,4,5] as well as to induce substantial deformational demands at beam-column joints, e.g., [6]. Alternative types of braces that can be used in a CBF and seem to exhibit a stable hysteretic behaviour is the three-segment brace recently proposed by Seker et al. [7] and the superelastic shape memory alloy (SMA) brace proposed by McCormick et al. [8]. A comparative study on the seismic performance of CBFs with SMA braces and BRBFs, has been also performed [9].

To avoid common brace buckling problems, CBFs with tension-only braces (employing steel rods) have been proposed [10–12] but their use seems to be restricted only in seismic retrofitting of existing structures [13–15]. On the other hand, application of tension-only braces in SCBFs is prohibited [2,3], whereas in [1], tension-only bracing type behaviour due to purely elastic buckling of the braces is mentioned but without any further recommendation. The use of tension-only braces (using spiral strand ropes or cables) in a seesaw configuration [16–18] seems to be a promising seismic force-resisting system but further research is demanded before its codification.

The purpose of this paper is to revisit the concept of the tension-only concentrically braced frames in an effort to recommend an improved version for them. The motivation behind the recommendation of tension-only braces is essentially to avoid buckling of the brace. This buckling avoidance is accomplished by means of a specific brace detailing that allows the brace to slide when in compression and to develop a resisting force when in tension. Removing brace buckling issues certainly improves the overall design process of concentrically braced frames and renders unnecessary any slenderness considerations related to their seismic behavior [19].

Slotted holes has been initially introduced in the seismic design of connections of CBFs by FitzGerald et al. [20] and Grigorian et al. [21]. They basically constitute modified bolted connections designed to dissipate energy through friction in both tension and compression. However, an unstable hysteretic behaviour can be met in slotted bolted connections due to problems associated with friction and wear between steel surfaces as well as with brittle failures when the bolt-shank impacts the end of the slot. Variants of slotted-bolted connections in

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https://doi.org/10.1016/j.soildyn.2018.08.017
Received 29 July 2018; Received in revised form 12 August 2018; Accepted 12 August 2018

braced frames have been proposed aiming to dissipate energy by straight-line or rotational sliding [22,23] and a remedy to bolt impact has been suggested [24]. Slotted-bolted connections have been also used in the sliding hinge joint moment connection [25], in the asymmetric friction connection [26] and in shear connections [27], whereas a detailed review regarding their ductile behaviour as well as their implementation in praxis can be found in [28–30]. Application of slotted-bolted connection in reducing the gusset plate-frame interaction of BRBFs has been very recently presented by Zhao et al. [31].

Focusing our interest in the implementation of slotted-bolted connections in steel CBFs [22,23], positioning the bolts in the middle of the slotted holes performed in one or more steel plates, permits energy dissipation through sliding. Sliding continues until bolts reach the end of the slotted holes, where a resisting force is developed. Taking into account that a steel brace exhibits both tension and compression during a seismic motions, the sliding joint details of [22,23] can be used. However, problems associated with the absence of a stabilizing compressive force under reversals of motions may occur.

The idea proposed in this paper is to employ the sliding joint concept but now by positioning the bolts (pins) directly at the one end of the slotted hole. The detail of this sliding joint is indicatively shown in Fig. 1. A high-strength pin slides along the slotted hole performed on a brace of hollow section (Fig. 1b). The brace bears cuts of a length *L* in order to be connected with the gusset plate that holds the pin (Fig. 1a and c). This way the steel braces work only in tension and compression cannot be developed as long as the clearance of the slotted hole is not exceeded by the sliding pin. The proposed brace detailing is considered to be cheaper and easier to fabricate in comparison with the corresponding detailing of other bracing systems, e.g., [32,33]. On the other hand, the impact of the pin to the end of a slotted hole is a matter of serious concern not only from the design point of view but most importantly from the fact that current seismic design codes do not accept or promote impact type of behavior in structures. Nevertheless, the proposed tension-only braces are studied herein in order to check if they can satisfy basic seismic response and demands indices when used in a steel structure. Additionally, a comparison of concentrically tension-only braced frames with BRBFs is performed. The use of the proposed tension-only braces as a seismic force-resisting system for steel structures is finally assessed.

2. Description of the steel structures under study

Three dimensional steel structures, used for office-residence purposes, having 4, 6 and 8 storeys (Type A) as well as a typical 2-storey industrial building (Type B) are selected for seismic response computations.

A typical floor plan view and front views for the 4- and 6-storey structures of Type A are shown in Figs. 2 and 3, respectively. Each bay has a span of 6.0 m, whereas the height of each storey is 3.0 m. The stressed black lines in Fig. 2 indicate the position of the braces. Type A and Type B steel structures are designed with tension-only braces or with buckling-restrained braces (BRBs).

The configuration of the braces can be in inverted V, diagonal and multistory X forms, as shown in Fig. 4. In that figure, the middle case of diagonal bracing corresponds to a different position of the braces in the frames of the perimeter from that shown in Fig. 2. Moreover, the symbols used, i.e., A4a, A4b, A4c mean that the structure under study corresponds to Type A, has 4 storeys and the brace configuration differs and may be *a* (inverted V), *b* (diagonal) or *c* (multistory X). Similarly one defines, A6a, A6b, A6c and A8a, A8b, A8c for the cases of 6- and 8-storey structures of Type A, respectively.

The floor plan of the Type B 2-storey structure is shown in Fig. 5. Each bay has a span of 6.0 m, and the height of each storey is 3.0 m, whereas the stressed black lines indicate the position of the braces. Only the B2a structure is studied which means that the inverted V configuration of Fig. 4 is employed.

Type A and B structures are designed according to EC3 [34] and EC8 [35] for the combinations: i) 1.35·dead load + 1.5·live load and ii) dead load + 0.3·live load + seismic load. In particular, dead and live loads on floors have been considered to be 8.0 kN/m^2 and 3.0 kN/m^2, respectively, whereas the seismic load is calculated using the design spectrum of EC8 [34] that corresponds to a PGA of 0.36 g and to a soil of class D. Fixed-based conditions are assumed and soil-structure-interaction effects are neglected, even though this is not realistic when soil of class D is considered. Behavior factors are conservatively considered to be equal to 2.5 for the *a* and *c* configurations and equal to 4.0 for the *b* configuration of Fig. 4. Effects of accidental torsion are also taken into account, even though, placing of braces on axis with the perimeter of the structures almost precludes torsional effects. Orientation of columns follows [36], forming, thus, a strong perimeter frame. Steel grade is S275.

Sections for beams and columns as well as the cross-sectional area of the core of the BRBs are shown in Table 1, whereas the corresponding sections of beams, columns and braces for the case of tension-only braced structures are shown in Table 2. In both tables, the symbols A4a etc., are previously explained. Figs. 6 and 7 display the sections at an exterior frame of the structure A6b having buckling-restrained and tension-only braces, respectively. More details regarding the design of the steel structures under study using BRBs can be found in [37]. The design of tension-only braces is performed using the analysis option for tension-only braces of SAP 2000 [38].

For the steel structures with BRBs, the design storey drift is considered to be 1.5% and the design axial displacement which the BRB should accommodate is two times this drift, i.e., 8.04 cm. For the steel structures with tension-only braces, the design drift is also 1.5% and, thus, the design slot clearance is 4.02 cm. All connections for steel structures with BRBs and tension-only braces are moment-resisting ones, except those of the BRBs that are pinned and those of the tension-only braces that are pinned but with axial translation free. The moment connections are expected to provide reserve strength and to reduce both the drift and the residual drift of the stories.

3. Structural modelling and seismic motions used

The steel structures having buckling-restrained and tension-only braces, are subjected to the 7 accelerograms of Table 3 and their seismic response is determined through non-linear time-history analyses using the computer analysis software RUAUMOKO 3D [39]. These accelerograms correspond to recordings of near-field strong ground motions because these type of ground motions have been repeatedly reported in the literature to produce large residual deformations in steel structures. The two horizontal components of these accelerograms are used interchangeably in both directions but their variation using an angle of incidence is not studied.

Diaphragm action is assumed at every floor due to the presence of a composite slab. Large deformation and second order effects are also taken into account [39] and an inherent viscous damping of 3% of critical is considered. Beams and columns are modelled using standard frame elements with concentrated plasticity assuming a strain hardening of 2%. The interaction of axial load with biaxial moment is considered for all columns. Column panel zone deformations as well as gusset plates are not modelled in the analyses performed herein and are left out for a future work.

The BRB model used for seismic response purposes should include an appropriate isotropic hardening law or a combination of isotropic and kinematic hardening [36,40,41]. This is particularly important when assessing the force demands imposed to beams and columns by the BRBs by non-linear time-history seismic analysis However, for reasons of conservativeness in the seismic response calculations performed herein, the BRB is modelled as an inelastic truss member [39] on the basis of the equivalent area [41]. The post-yield stiffness of the BRB core is assumed to be 2% of the axial elastic stiffness. A nominal

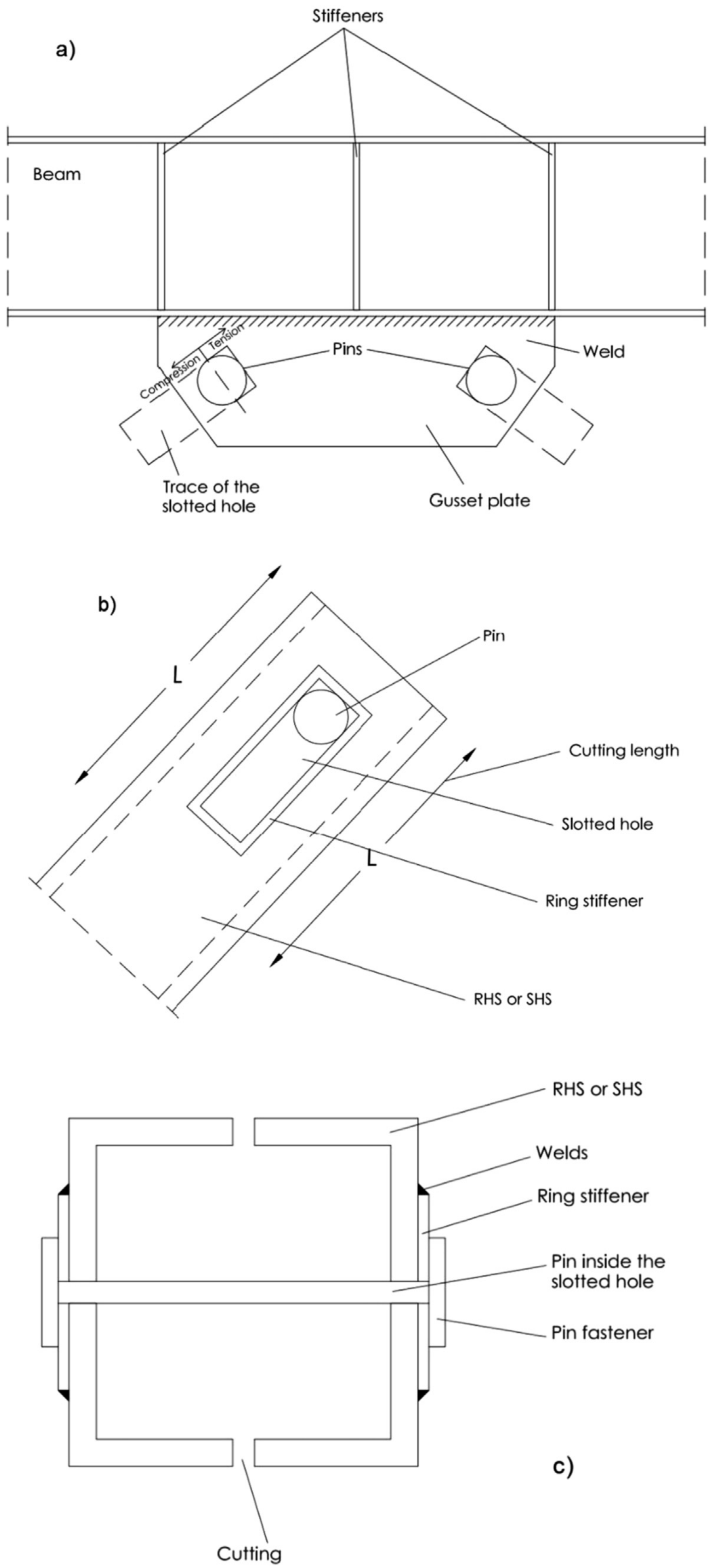

Fig. 1. - Detail of the proposed sliding joint that creates a tension-only brace.

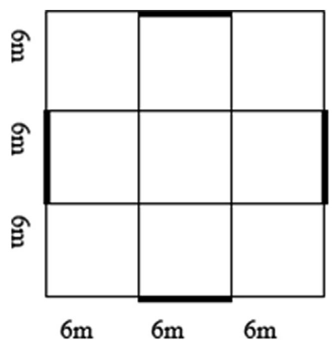

Fig. 2. - Floor plan of Type A structures and location of the braces.

yield strength of 245 MPa is assumed for the BRB core in order to acknowledge the variability of yield stress in the material of the core.

Modeling of tension-only braces is performed using the IHYST = 5 model of RUAUMOKO [39] which is shown in Fig. 8. According to that model an initial gap (slackness) equal to the slot clearance is considered in compression, whereas in tension the corresponding gap is zero. Strain-rate related constants are set to zero [39].

4. Results from seismic analyses

Due to the symmetry of the steel structures examined, the seismic analyses results presented in this section involve only the unfavorable structural responses. Interchangeability of the two horizontal components of the accelerograms is taken into account when these unfavorable cases are sought.

Mean peak interstorey drift ratios (IDR), peak residual interstorey drift ratios (RIDR) and peak floor accelerations (PFA) estimates from the non-linear inelastic time-history seismic analyses are shown. In all seismic analyses performed, the BRBs yielded first at small drift levels of about 0.5%, whereas the design tensile strength of the tension-only braces is never exceeded. Even though beams and columns are designed with an overstrength [35], actual seismic demands may force significant yielding to them due to the additional forces induced by the braces. Therefore, mean seismic demands in terms of: i) axial force and rotation in columns, ii) shear force and bending moment at the beam where the BRBs and the tension-only braces are connected, and iii) axial displacement of BRBs and of tension-only braces are also computed.

The seismic response and demands results presented and discussed in the following involve only the steel structures having the brace configuration *a*, i.e., the inverted V configuration. Similar results in a qualitative sense are found for the rest of the structures, with different brace configurations, of Table 1, even though they are not presented herein due to space limitations. Thus, in all figures that follow, abbreviations of the forms 'A4aT' and 'A4aBRB' mean the A4a structures having tension-only braces and BRBs, respectively.

Modal analysis results are firstly shown in Table 4, demonstrating that the first two natural periods of the structures A4a, A6a, A8a and B2a are very close. This period coincidence is representative of the structures studied herein and cannot be generalized.

Figs. 9–11 display the peak IDR values for the case of A4a, A6a and A8a structures. From these figures, it can be concluded that large peak IDRs up to about 2.5% may be exhibited, surpassing, thus, the design value of 1.5%. Smaller peak IDRs than the design value of 1.5% are found for the structure B2a, as shown in Fig. 12. In general, structures A4a, A6a, A8a and B2a having tension-only braces exhibit the same IDR pattern with those having BRBs, with only difference being that the IDR values for the case of tension-only braces are slightly higher in comparison to those of BRBs.

Figs. 13–15 display the peak RIDR values for the case of A4a, A6a and A8a structures. From these figures, it can be concluded that large peak RIDRs up to about 0.7% may be exhibited, surpassing, thus, the threshold value of 0.5% [36]. This essentially means, that significant structural damage is expected and the structure is practically non-usable. Smaller RIDRs are found for the structure B2a, as shown in Fig. 16. In general, structures A4a, A6a and A8a having tension-only braces exhibit the same RIDR pattern with those having BRBs, and with the exception of structure A8a, RIDR values for the case of tension-only braces are slightly higher in comparison to those of BRBs.

The range of peak ground acceleration (PGA) for the strongest component of the accelerograms of Table 3 is 0.37–0.70 g. In view of that, the ratio of PFA to PGA along height is shown in Figs. 17–20 for the structures A4a, A6a, A8a and B2a, respectively. From these figures, it can be concluded that PFA/PGA values in the range of 0.5–1.6 g may be exhibited, where less than unity values for PFA/PGA mean that the tension-only braces or the BRBs can limit the accelerations imparted to structure. However, amplifications of accelerations, i.e., PFA/PGA values above unity, may also occur at some floors. In general, structures A4a, A6a, A8a and B2a having tension-only braces exhibit an almost similar PFA/PGA pattern with those having BRBs, and with the exception of structure A8a, PFA/PGA values for the case of tension-only braces are slightly lower in comparison to those of BRBs.

Seismic demands in columns strongly depend on the extent and pattern of yielding occurring during the earthquake. Considering the most heavily stressed column for A4a, A6a, A8a and B2a structures, Figs. 21–24 display the height wise ratio of the axial force N to the design axial strength N_d. From these figures, it can be concluded that this ratio attains higher values for the structures having tension-only braces in comparison to those having BRBs. Nevertheless, values of this ratio in excess of 0.40–0.50 in conjunction with biaxial moment interaction should be view with extreme caution because the deformational capacity of the columns may be significantly affected by flange and web

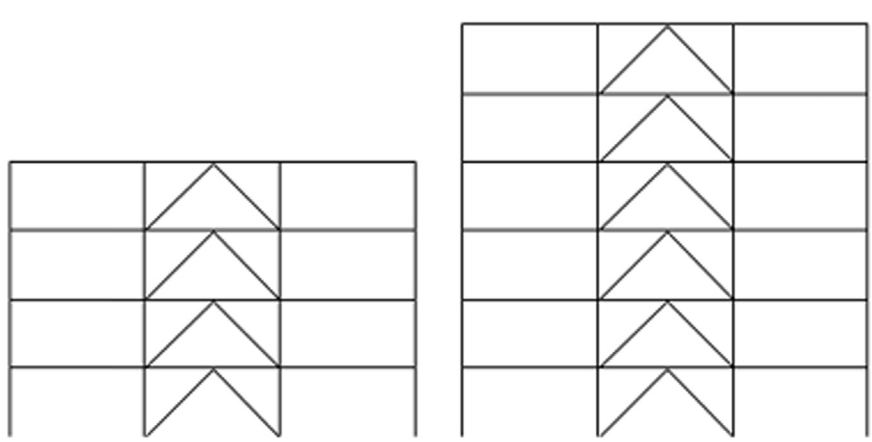

Fig. 3. - Front views of 4- and 6-storey Type A structures.

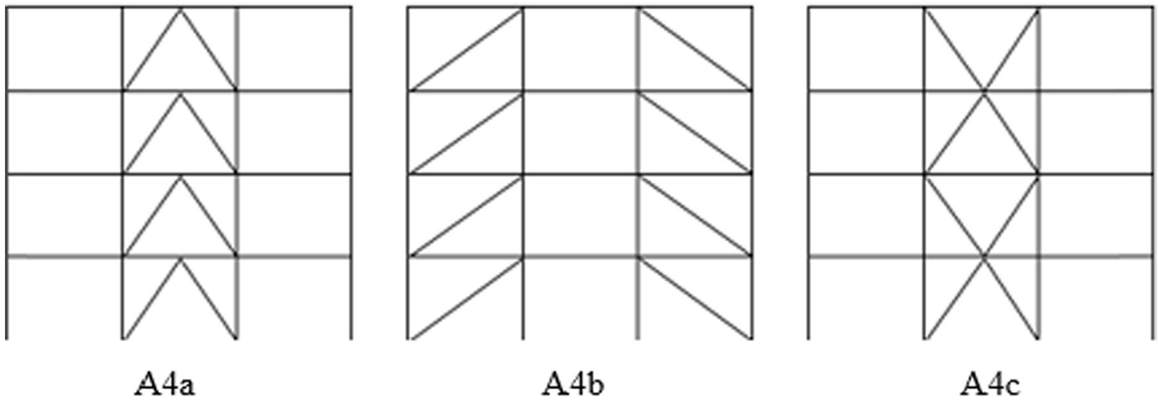

Fig. 4. - Configuration of braces for Type A structures.

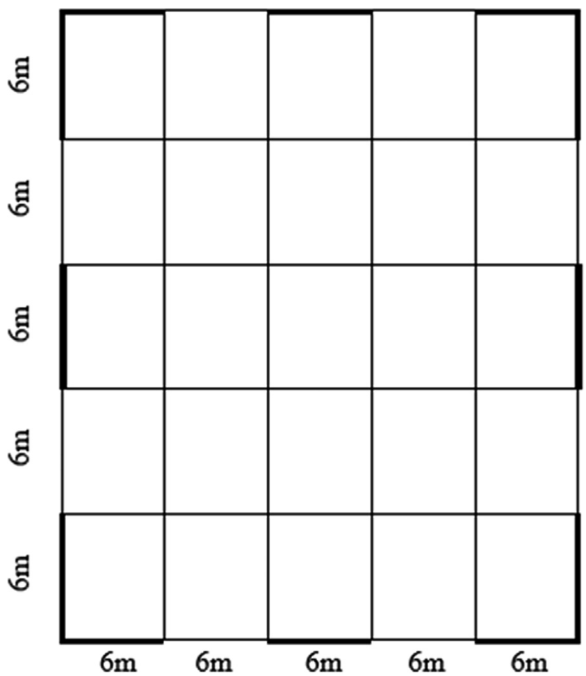

Fig. 5. - Floor plan of Type B structure and location of the braces.

Table 1
Design of Type A and B structures with BRBs.

Structure	Columns	Beams	Cross sectional area of the BRB core (cm^2)
A4a	HEB 320	IPE 220	52
A4b	HEB 320	IPE 220	52
A4c	HEB 320	IPE 220	52
A6a	HEB 400	IPE 240	70
A6b	HEB 400	IPE 240	70
A6c	HEB 400	IPE 240	70
A8a	HEB 500	IPE 270	87
A8b	HEB 500	IPE 270	87
A8c	HEB 500	IPE 270	87
B2a	HEB 500	IPE 270	140

Table 2
Design of Type A and B structures with tension-only braces.

Structure	Columns	Beams	Braces
A4a	HEB 320	IPE 220	SHS 80 × 80 × 5
A4b	HEB 320	IPE 220	SHS 80 × 80 × 5
A4c	HEB 320	IPE 220	SHS 80 × 80 × 5
A6a	HEB 400	IPE 240	SHS 80 × 80 × 5
A6b	HEB 400	IPE 240	SHS 80 × 80 × 5
A6c	HEB 400	IPE 240	SHS 80 × 80 × 5
A8a	HEB 500	IPE 270	SHS100 x 100 × 7.1
A8b	HEB 500	IPE 270	SHS100 x 100 × 7.1
A8c	HEB 500	IPE 270	SHS100 × 100 × 7.1
B2a	HEB 500	IPE 270	SHS 80 × 80 × 5

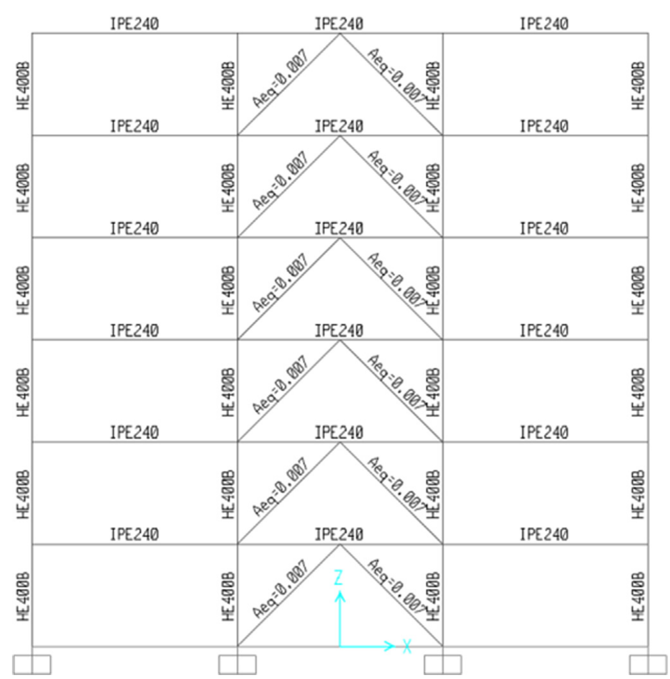

Fig. 6. - Sections at an exterior frame for the A6a structure with BRBs.

local buckling. Thus, the ratio N/N_d is crucial for both types of braces studied herein and of course even more crucial for the case of tension-only braces. Maximum inelastic column end rotational demands computed are 0.07 rads and 0.076 rads for the structures having BRBs and tension-only braces, respectively, but a global failure of columns is not indicated.

Considering the beam where the BRBs and the tension-only braces are connected to be adequately braced against lateral torsional buckling, Table 5 presents its seismic demands in terms of shear force and bending moment for the cases of structures A4a, A6a and A8a. These shear force and bending moment demands are normalized by their corresponding design strengths. From these results, it can be concluded that the BRB-beam or the tension-only brace-beam may fail due to flexure, especially when tension-only braces are connected, whereas shear force demands are low. At this point it should be mentioned that this beam did not exhibit failure in flexure for the case of

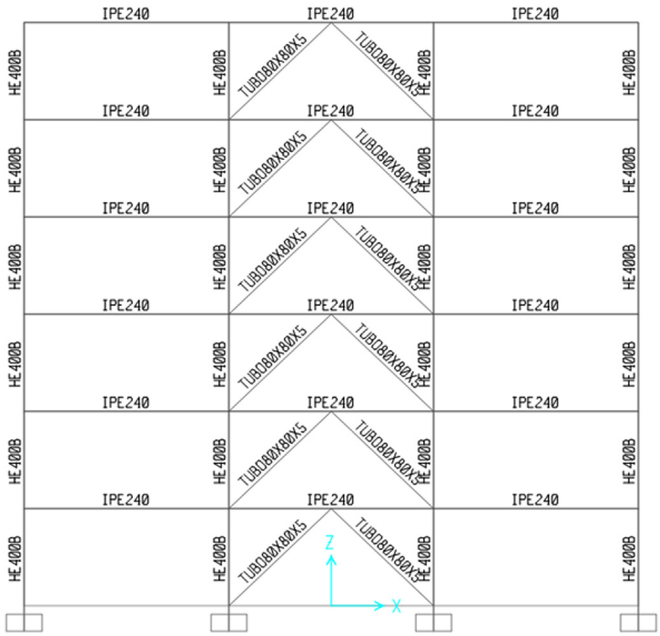

Fig. 7. - Sections at an exterior frame for the A6a structure with tension-only braces.

Table 3
Accelerograms used.

Recording station	Earthquake	Year – Country
El centro array 05	Imperial valley	1979 – U.S.A
Meloland route overpass	Imperial valley	1979 – U.S.A
Parachute test site	Superstition hills	1987 – U.S.A
Lucerne valley	Landers	1992 – U.S.A
Sylmar converter station	Northridge	1994 – U.S.A
Takarazuka	Kobe	1995 – Japan
Lixouri	Kefalonia	2014 – Greece

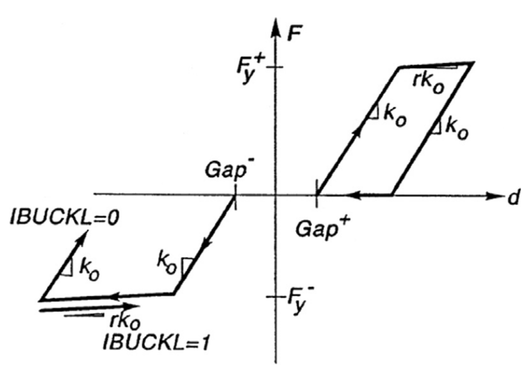

Fig. 8. – The IHYST = 5 model of RUAUMOKO [39].

Table 4
Periods of the first two modes for steel structures A4a, A6a, A8a and B2a.

Structure	1st mode (s)	2nd mode (s)
A4a – BRB	1.26	0.82
A4a – Tension-only brace	1.30	0.83
A6a – BRB	1.92	1.25
A6a – Tension-only brace	1.96	1.26
A8a – BRB	2.21	1.45
A8a – Tension-only brace	2.25	1.47
B2a – BRB	0.30	0.24
B2a – Tension-only brace	0.31	0.24

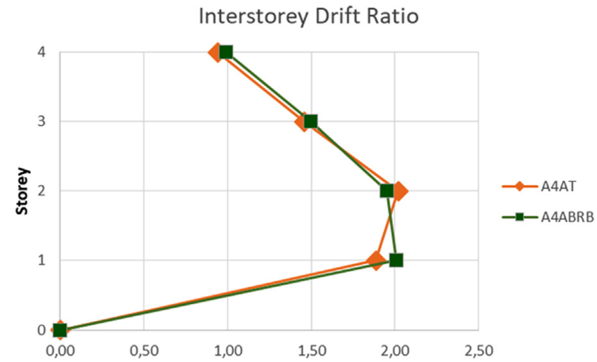

Fig. 9. – Mean peak IDRs for A4a structures.

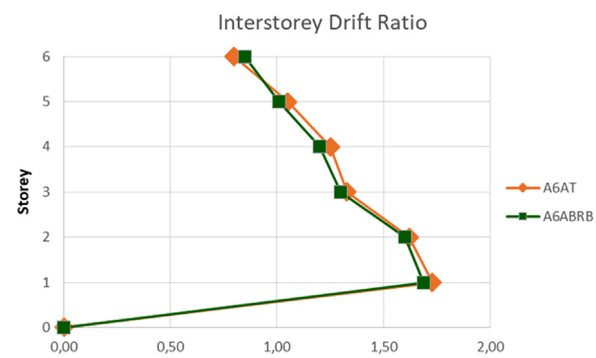

Fig. 10. – Mean peak IDRs for A6a structures.

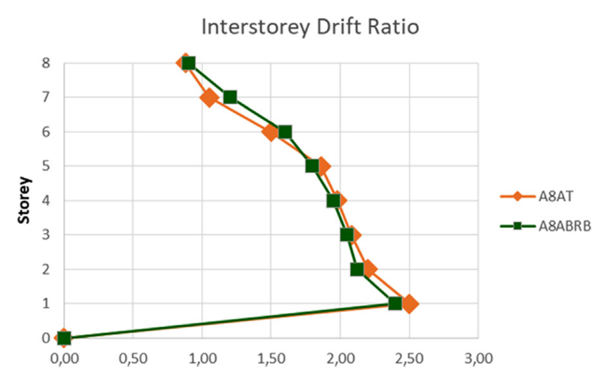

Fig. 11. – Mean peak IDRs for A8a structures.

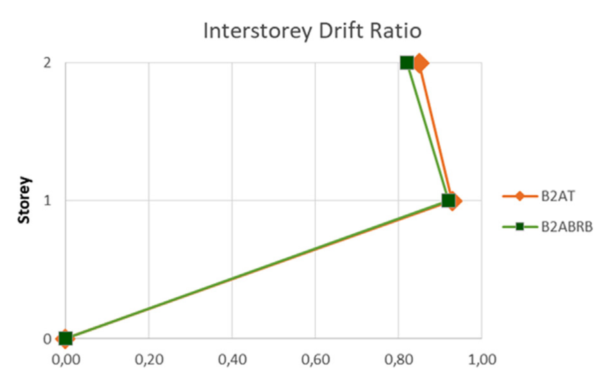

Fig. 12. – Mean peak IDRs for B2a structures.

diagonal and multistory X-bracing configurations.

Finally, it is a matter of concern if brace compression is activated when the design slot clearance is exceeded. Therefore, mean values for the axial displacement of the tension-only braces along with the

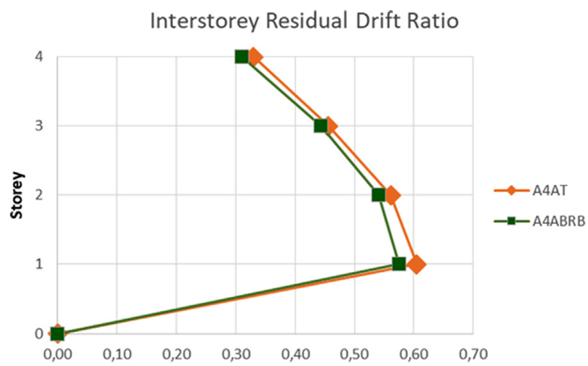

Fig. 13. – Mean peak RIDRs for A4a structures.

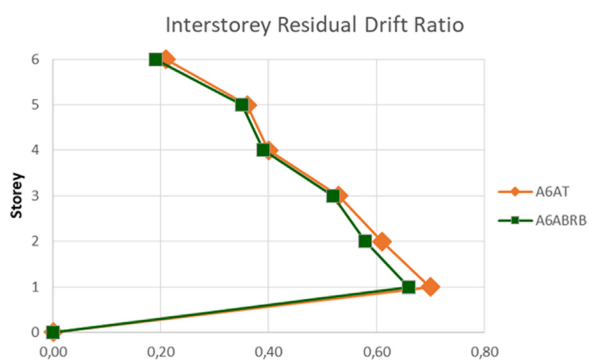

Fig. 14. – Mean peak RIDRs for A6a structures.

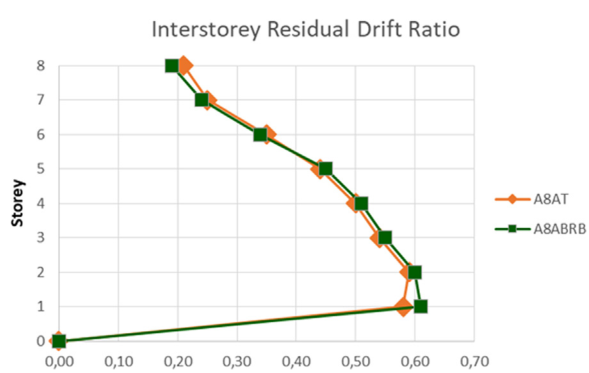

Fig. 15. – Mean peak RIDRs for A8a structures.

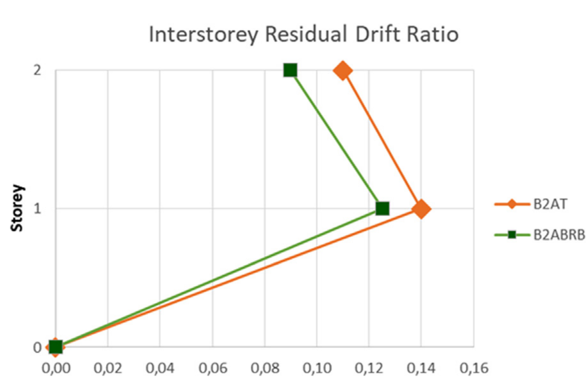

Fig. 16. – Mean peak RIDRs for B2a structures.

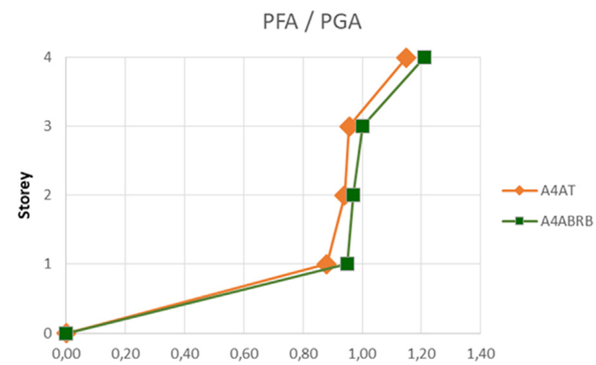

Fig. 17. – Mean peak PFAs for A4a structures.

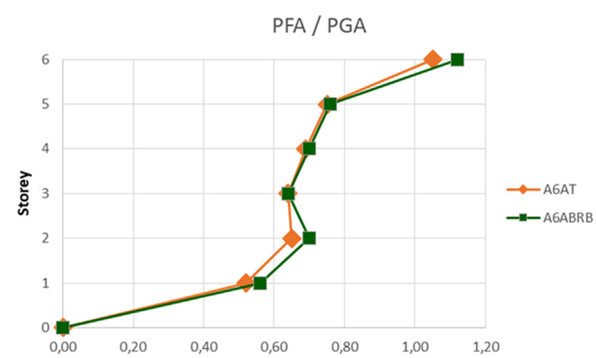

Fig. 18. – Mean peak PFAs for A6a structures.

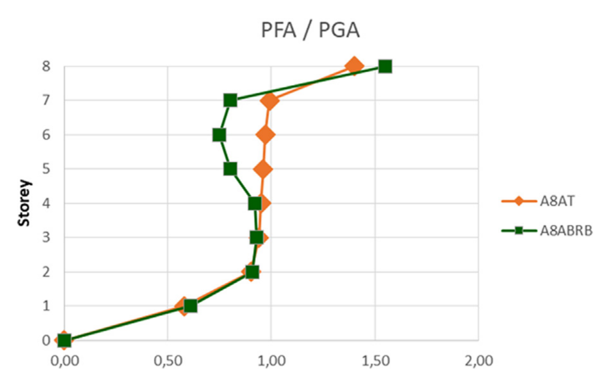

Fig. 19. – Mean peak PFAs for A8a structures.

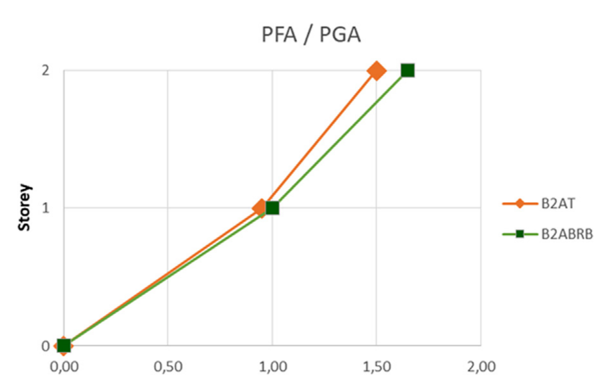

Fig. 20. – Mean peak PFAs for B2a structures.

corresponding ones for the BRBs and are presented in Table 6 for A4a, A6a and A8a structures. These axial displacements are almost similar for these two types of braces and their design values, mentioned in Section 2, are not exceeded.

5. Conclusions and future needs

In this paper, the concept of tension-only concentrically braced steel structures is revisited and it is shown that common steel braces can be

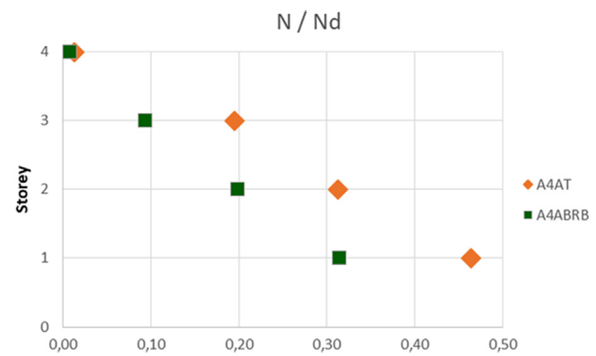

Fig. 21. – Column seismic demands in terms of N/N_d for A4a structures.

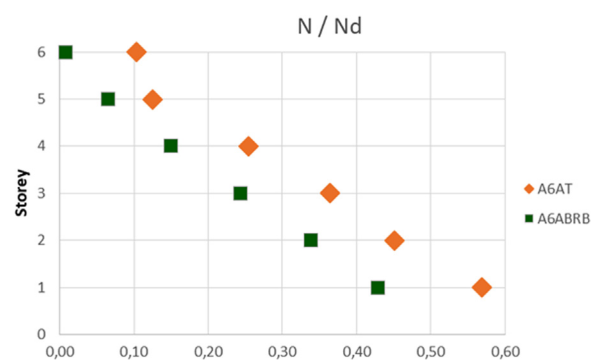

Fig. 22. – Column seismic demands in terms of N/N_d for A6a structures.

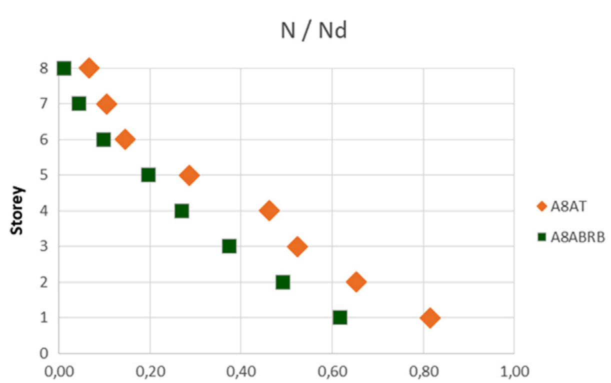

Fig. 23. – Column seismic demands in terms of N/N_d for A8a structures.

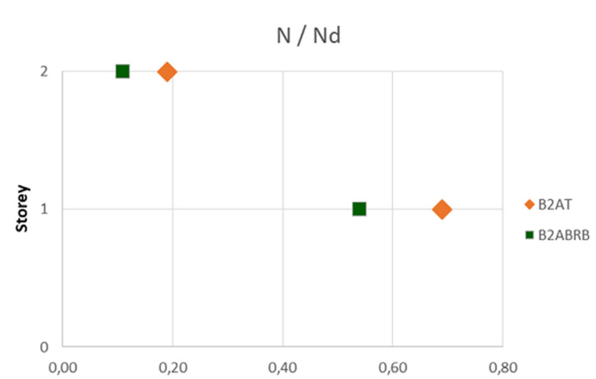

Fig. 24. – Column seismic demands in terms of N/N_d for B2a structures.

detailed to sustain only tension and no compression. Therefore, usual buckling problems associated with steel braces are eliminated. The proposed brace detailing is considered to be cheap and easy to fabricate.

Table 5
Mean demand ratios for the beam connected with the braces.

	BRB			Tension-only brace		
Demand ratio	A4a	A6a	A8a	A4a	A6a	A8a
Shear	0.196	0.215	0.247	0.198	0.215	0.166
Moment	0.947	0.971	0.968	0.955	0.972	1.439

Table 6
Mean axial displacement values for BRBs and tension-only braces.

	BRB (cm)	Tension–only brace (cm)
A4a	3.18	3.09
A6a	3.05	3.04
A8a	2.86	2.86

Various seismic response and demands results are presented and from them it can be generally assessed that tension-only braced structures have a comparable behavior to the BRB structures. As holds for BRB structures, tension-only braced structures face the issue of significant residual displacements. The threshold RIDR value of 0.5%, accepted by the engineering community, is surpassed for both types of structures. But for the tension-only braced structures, even more important is the overstress in compression induced to the columns of the lower storeys, something that the present work highlights but cannot, at the time being, propose a remedy for it. On the other hand, an appropriate slot clearance ensures that compression to the tension-only braces is not developed.

The level of the friction forces needed to be surpassed before sliding is not modeled, but it is deemed that its contribution is small and does not substantially alter the seismic response results obtained herein. However, the impact of the pin to end of the slotted hole when tension is developed is a matter of further investigation. A hardening or a quenching procedure for the steel surfaces near to the end of the slotted holes in conjunction with a detailed finite element analysis is suggested in order to assess the stress field around the slot during impact. Taking also into account, that the philosophy of current seismic design codes precludes any recommendation for impact forces in the design of conventional (i.e., those that are not equipped by energy dissipation devices or by other kinds of seismic fuses) structures, the aforementioned investigation on impact forces, even it is mandatory, may not be easily accepted by the engineering community.

Finally, on the basis of studying and properly designing the slotted holes against impact forces, the possibility of assigning a small but controllable compression to the braces, leading thus, to the concept of tension-dominant braces, may be proven to be a good solution regarding reduction of RIDRs and of the overstress of columns in compression. In this direction, the use of ellipsoidal instead of straight-line sliding may be also needed.

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