

Please cite as: Málaga-Chuquitaype, C., Elghazouli, A.Y., Enache, R. (2015) Contribution of secondary frames to the mitigation of collapse in steel buildings subjected to extreme loads, *Structure and Infrastructure Engineering*, doi: 10.1080/15732479.2014.994534

RESEARCH ARTICLE

Contribution of secondary frames to the mitigation of collapse in steel buildings subjected to extreme loads

C. Málaga-Chuquitaype^a * and A.Y. Elghazouli^a ¹ and R. Enache^a ²

^a*Department of Civil & Environmental Engineering, Imperial College London, UK
(Received 00 Month 200x; final version received 00 Month 200x)*

This paper examines the contribution of secondary (or gravity) frames to the mitigation of collapse in steel buildings subjected to extreme loading conditions arising from multiple hazards. By considering sidesway and vertical mechanisms as representative of most building collapse modes, this study evaluates the effects of various secondary-frame parameters on the overall building collapse capacity. Given that the beam-to-column joint response typically dominates the behaviour of secondary frames, particular emphasis is given to key connection response characteristics such as stiffness, strength and ductility. The paper starts with a brief description of the probabilistic assessment framework that serves as the basis for the study followed by the assessment of the influence of partially-restrained (PR) gravity frames on the collapse capacities of steel structures by means of generalized structural sub-assemblages. Secondary frames are found to provide up to a three-fold increase in the sidesway collapse resistance depending on the magnitude of second order effects. Likewise, the vertical (progressive) collapse capacity is greatly affected by the connection depth versus beam length ratio and the secondary connection strength. A brief discussion on directions for the provision of simplified approaches for the design of secondary frames is presented and illustrated through two case studies involving steel frame buildings subjected to earthquake and blast scenarios, as well as localized fire and blast hazards. This study offers a fundamental step towards the provision of design and assessment strategies that incorporate secondary systems into a multi-hazard structural evaluation framework.

Keywords: extreme loading; sidesway collapse; progressive collapse; gravity frames; partially-restrained connections

* Corresponding author. Email: c.malaga@imperial.ac.uk

¹ Email: a.elghazouli@imperial.ac.uk

² Email: r.enache12@imperial.ac.uk

1. Introduction

Accidental or malicious actions (e.g. fires, explosions, terrorist attacks, etc.) as well as earthquakes and other severe natural hazards could cause local failure mechanisms that can propagate throughout the building and trigger partial or total collapse. The mitigation of risks associated with multiple hazards by means of proper assessment and design practices constitutes the main objective of structural design under such scenarios. Traditionally, collapse mitigation strategies have focused on the effects of individual hazards. However, there is a shift towards the consideration of multiple-hazards into structural design and assessment methodologies (Li *et al.*, 2012). Furthermore, given that public safety is the governing factor in structural design, the probability of collapse constitutes a suitable metric for multi-hazard risk assessment (Ellingwood, 2006).

Although all structures are susceptible to collapse to some extent (Ellingwood and Leyendecker, 1978), the risk from abnormal loading conditions is particularly relevant for strategic facilities. Such facilities will also be subjected to more than one hazard with the potential to cause collapse during their service-life and would subsequently need additional engineering guidance. This fact is recognized by international design provisions which demand a systematic risk assessment of structural robustness for high-consequence class structures (CEN, 2002a, ODPM, 2004, GSA, 2003, DOD, 2005). However, virtually no guidance is provided on how such assessment is to be undertaken, and only limited advice is offered regarding the management of multiple risks and the associated mitigation strategies. Therefore, there is a need to assess the effects of various extreme loading conditions on buildings with a focus on structural collapse and the evaluation of alternative strategies for enhancing their structural integrity under multiple hazards.

An adequate design of secondary frames taking into account different collapse scenarios, has the potential for improving the overall building performance. Secondary systems are defined by current seismic design standards as those structural systems contributing typically less than 15% in terms of the elastic lateral stiffness of the building (CEN, 2005a). These provisions aim to equip buildings with a primary system of adequate lateral strength and ductility to withstand extreme loads while disregarding secondary frames which are designed to carry gravity loads alone. However, it has been shown that, depending on their strength and stiffness, secondary systems can modify the post-elastic response of a structure (Vargas and Bruneau, 2006). Previous research (Hsiao *et al.*, 2013, Málaga-Chuquitaype and Elghazouli, 2010a) has also highlighted the positive effects that gravity frames can have on the seismic performance of steel structures. Similarly, the potential benefits of an adequate design of secondary frames have been recognised in the course of examining residual displacements in buckling restrained braced frames (Kiggins and Uang, 2006, Pettinga *et al.*, 2007). Nevertheless, no systematic evaluation of the contribution of secondary frames to the mitigation of the most common forms of structural collapse has been undertaken from a multi-hazard perspective.

In light of the above discussion, this paper provides insight into the contribution of secondary frames to the mitigation of total collapse probabilities in steel buildings subjected to extreme loading scenarios arising from multiple hazards. To this end, an assessment of the influence of key secondary system parameters such as stiffness, strength and duc-

tility on the inelastic demands of steel-framed structures is presented. The paper starts with a description of the adopted probabilistic framework that serves as the basis for the comparisons, as well as the numerical tools employed for the estimation of the structural response. Subsequently, the contribution of secondary systems in enhancing the sidesway and vertical collapse capacities of steel buildings is analysed by means of simplified structural sub-assemblages. It is shown that the secondary system stiffness, strength and their corresponding ductility have a considerable influence in enhancing (or sometimes reducing) the collapse capacity of steel structures. In particular, secondary frames are found to provide up to a three-fold improvement in the sidesway collapse capacity depending on the magnitude of second-order effects. Similarly, the vertical (progressive) collapse capacity is greatly affected both positively and negatively by the secondary connection depending on its strength and ductility. It is noted that very large values of secondary connection strength to beam plastic capacity can limit the overall collapse resistance due to reductions in the joint ductility associated with stronger connections. It is recommended that a combined assessment of the interdependencies between connection stiffness, capacity and ductility should be carried out in light of the levels of risk associated with progressive and sidesway collapse. The application of these findings is illustrated through two case studies involving a six-storey concentrically-braced frame subjected to blast and seismic hazards as well as a four-storey moment-resisting frame analysed under localized fire and blast scenarios. The investigation demonstrates that secondary systems (which may not have a notable contribution in terms of lateral load-carrying capacity) can play a significant role in mitigating the probability of dynamic instability under multiple hazard scenarios, and that a probabilistic comparison of different design alternatives is within the reach of practical assessment and design.

2. Collapse assessment framework

In order to establish the degree of contribution of secondary frames to the mitigation of collapse due to extreme loading caused by multiple hazards, both a faithful structural model and an effective probabilistic approach that takes into account the uncertainties in the estimation of collapse loads are required. Besides, a quantitative and objective measure of the risk is also desirable. To this end, the annual probability of structural collapse (Ellingwood, 2006) due to a critical event (E) can be approximated by:

$$P[\text{Collapse}] = P[\text{Collapse}|D]P[D|E]\gamma_E \quad (1)$$

where D is the event that structurally significant damage results and γ_E is the annual mean rate of occurrence of E . Similarly, if multiple hazards and damage states are possible, Equation 1 can be generalized as (Ellingwood, 2006):

$$P[\text{Collapse}] = \sum_E \sum_D P[\text{Collapse}|D]P[D|E]\gamma_E \quad (2)$$

Finally, assuming that ν_m represents the risk level below which society normally does not impose regulatory guidance (i.e *de minimis* risk), alternative risk mitigation strategies can be examined in light of the following requirement:

$$\sum_E \sum_D P[\text{Collapse}|D]P[D|E] < \nu_m/\gamma_E \quad (3)$$

It is important to note that the above formulations are based on the total probability theorem and hence assume collectively exhaustive and mutually exclusive events. Therefore, extreme loading conditions arising from hazard interactions (e.g. fires following earthquakes, etc.) are outside the scope of the present study. However, given the positive correlation observed between structural rehabilitation techniques for competing hazards (Li *et al.*, 2012), the general trends found in this paper are expected to apply in some hazard-interaction cases.

Two main collapse mechanisms are assumed to be representative of most building collapse modes herein, namely: i) sidesway collapse due to lateral earthquake action and ii) vertical (progressive) collapse due to loss of the vertical load carrying capacity in one or more structural elements. Therefore, the risk associated with sidesway seismic collapse can be estimated by integrating the structural seismic collapse fragility, obtained from Incremental Dynamic Analyses (IDA) (Vamvatsikos and Cornell, 2002), with the annual rate of exceeding spectral acceleration levels, obtained from a Probabilistic Seismic Hazard Assessment, as suggested by Ibarra and Krawinkler (2005). To this end, sidesway collapse is defined herein as the stage of dynamic instability in which a very small increment in the intensity of the ground motion leads to disproportionately large increments in the global deformation demands. This method assumes that a given set of ground-motions can be scaled to represent different hazard levels, and the collapse fragility curve can be expressed as:

$$P[\text{Collapse}|IM = im_i] = P[IM_c < IM = im_i] \quad (4)$$

where IM represents the ground-motion intensity measure and the collapse capacity IM_c is the ground motion intensity at which dynamic instability occurs. Figure 1 illustrates the process by which the sidesway collapse capacity of a building (Málaga-Chuquitaype and Elghazouli, 2010a) is calculated. In this figure, each black line represents the numerically obtained relationship between ground-motion IM (typically Peak Ground or Spectral Acceleration in $[g]$) and a selected engineering demand parameter (EDP). This EDP is selected to represent the global response of the building and in this study is defined as the peak inter-storey drift along the height of the structure. The markers presented in Figure 1(b) represent the ordinate of the last point on the IDA curve before dynamic instability and therefore mark the collapse capacity associated with each individual ground motion. Based on such data points, a log-normal distribution can be fitted (as shown in Figure 1) to represent the probability distribution of collapse (Ibarra and Krawinkler, 2005).

Similarly, element removal scenarios are considered here as a suitable event-independent collapse assessment tool (Izzuddin *et al.*, 2008, Vlassis *et al.*, 2008) for vertical (progressive) collapse modes. The probability of collapse given an event, E (e.g. blast, fire, etc.), with potential to cause the loss of integrity in one or more structural elements can be assumed as:

$$P[\text{Collapse}|E] = \int I_{\text{Collapse}|E}(\phi)p(\phi)d(\phi) \quad (5)$$

where ϕ represents the uncertain quantities of interest (e.g. blast location and charge, fire location and burning time, etc.) that can be generated by means of Monte Carlo simulation procedures as proposed by Asprone *et al.* (2010) for the case of blast induced

collapse; $p(\phi)$ is the probability density function of ϕ and $I_{Collapse|E}$ is an index which is equal to 1 if ϕ leads to collapse or 0 otherwise. In this case, the event of collapse can be evaluated by means of Push-down Analysis which can be performed statically or dynamically in an incremental manner (Kim *et al.*, 2009). As an example, Figure 2 presents the calculation of the vertical collapse capacity of a structural system through Incremental Dynamic Push-down Analysis (IDPA). First, the dynamic response of the structure under the instantaneous removal of a main load carrying element is calculated for increasing vertical loads quantified by the load factor (Λ). Subsequently, an IDPA curve that relates Λ with the corresponding peak displacements is obtained. On this basis, the load factor at collapse (Λ_c) is identified as the factor by which the vertical loads should be multiplied in order to cause a progressive collapse mechanism (i.e. a plateau in the right hand side of Figure 2) for a given realization of ϕ . It is important to note that the collapse risk calculation for structures with partially-damaged elements is outside the scope of this paper. Similarly, non-zero initial conditions and damage on adjacent elements are ignored within the column removal scenario approaches.

3. Assessment of secondary frame contribution

This section examines the collapse response of generalized structural sub-assemblages with a view to evaluating the secondary system potential for enhancing the collapse capacity in steel buildings. The structural sub-assemblages considered herein are representative of typical steel buildings and incorporate a section of the primary frame together with the corresponding contributing portion of secondary frame. Based on the Total Probability Theorem as expressed in Equation 2, the effects of secondary frames on mitigating sidesway and progressive collapse are studied independently in this section. Subsequently, an evaluation of the overall effects of secondary systems on multi-storey structures subjected to concurrent hazards is presented in the next section. Given that accurate modelling of the connection behaviour including cyclic deterioration and possible dynamic effects is essential for the simulation of structural collapse, and considering that the response of secondary (or gravity) steel frames is largely controlled by the joint behaviour, the connection mechanical models employed in this study are first introduced below.

3.1 Numerical modelling of connections

This study employs two modelling approaches to represent the response of typical secondary frame connections under extreme loading as depicted in Figure 3: (i) mechanical-based component models (for the case of vertical collapse modes) and (ii) plastic hinge models (for sidesway seismic action). In both cases, an incremental procedure (Swanson and Leon, 2001, Málaga-Chuquitaype and Elghazouli, 2010b) for the estimation of the backbone force-displacement relationship of the whole connection is adopted and the stiffness of each equivalent T-stub or bolt-row (k_i) is calculated on the basis of the contribution of individual components. Once the joint backbone curve is estimated, a single rotational spring model that considers the full hysteretic response as observed in Figure 3(c) can be assembled on the basis of the parameters obtained by means of the more detailed multi-spring component model (Figure 3(b)). A typical

comparison between the predicted moment-rotation backbone curve and experimental results (Elghazouli *et al.*, 2009) is presented in Figure 4(a) whereas Figure 4(b) depicts the comparison of hysteresis curves for the same connection assembled by means of the Hysteretic Material model available in the OpenSees framework (McKenna, 1997). As demonstrated from Figure 4, the mechanical model predictions are in close agreement with the experimental results. The model is able to represent the key response parameters and features of the connection, namely the initial stiffness, connection capacity, post-yield hardening and onset of membrane effects at large displacements. In particular, the initial stiffness and capacity can be predicted within an accuracy of under 5% in all cases (Malaga-Chuquitaype and Elghazouli, 2010b). On the other hand, the differences between the experimental results and the corresponding backbone are due to the nature of the bi-linear idealisation employed which offers a reasonable trade-off between computational expense and advanced modelling features such as stiffness and strength degradation phenomena which are essential for the determination of collapse capacities (Lignos and Krawinkler, 2011).

In the case of idealised element removal scenarios, the downward movement of the unsupported building sections can cause the bending moments and axial forces acting on the adjacent connections to increase very rapidly and may eventually lead to their failure. To account for these effects, a modified bolt-row response that incorporates possible strain-rate effects is employed (Málaga-Chuquitaype *et al.*, 2014). To this end, the steel strength values employed in the calculation of the stiffness and capacity limits are modified by a Dynamic Increase Factor as proposed by Malvar and Crawford (1998):

$$\begin{aligned}\Phi_y &= \left(\frac{\dot{\epsilon}}{1 \times 10^{-4}} \right)^{\alpha_y} \\ \alpha_y &= 0.074 - 0.040 \frac{f_y}{414}\end{aligned}\tag{6}$$

$$\begin{aligned}\Phi_u &= \left(\frac{\dot{\epsilon}}{1 \times 10^{-4}} \right)^{\alpha_u} \\ \alpha_u &= 0.019 - 0.009 \frac{f_y}{414}\end{aligned}\tag{7}$$

where Φ_y and Φ_u are the dynamic increase factors for yield strength and ultimate strength, respectively, $\dot{\epsilon}$ is the strain rate and f_y is the nominal yield strength of steel. Figure 5 presents the force-displacement relationships obtained from dynamic tests (Málaga-Chuquitaype *et al.*, 2014) as well as their corresponding multi-spring mechanical model predictions. A constant strain rate equal to the peak rate observed during the test (i.e. 20 s^{-1}) (Málaga-Chuquitaype *et al.*, 2014) was assumed for the component-model calculations. It is evident from Figures 5(a) and 5(b) that the experimental behaviour is closely depicted by the mechanical model including reasonable estimations of post-elastic stiffness. Although some observed discrepancy in the response is apparent at large rotation levels (larger than 35 mrad), which can be attributed to modelling idealization of the material hardening, the capacities at failure are well predicted with an accuracy of $\pm 6\%$ in both cases.

It is important to note that the multi-spring models employed in the progressive collapse analyses performed in this study do not explicitly account for shear deformations. However, it has been recognized that shear failure is not likely to occur prior to flexural failure in connections subjected to column loss since the joints are usually designed with sufficient over-strength to resist increased shear forces (Fang *et al.*, 2012). Nevertheless,

in the case of localized fire, the joint shear resistance can be greatly reduced due to thermal effects. In order to consider these effects a capacity-related rigid-plastic shear response was employed herein following the approach proposed by Fang *et al.* (2012) which allows for shear force redistribution among adjacent connections. To this end, failure was assumed to occur whenever the total shear force applied onto the joint exceeded its overall shear capacity at elevated temperature.

3.2 Mitigation of sidesway collapse

Figure 6(a) shows the structural sub-assembly employed to assess the influence of secondary systems on the sidesway collapse capacity of concentrically-braced frame (CBF) buildings due to seismic action. Pin-connected rigid elements are utilized as beams and columns. The model was developed in Opensees (McKenna, 1997) and includes geometric and material non-linearities. The braces were represented by means of inelastic force-based buckling fibre elements (Dicleli and Mehta, 2007, D'Aniello *et al.*) and fatigue fracture was incorporated following the approach proposed by Uriz and Mahin (Uriz2008). The plastic hinge approach described above was employed to simulate the beam-to-column connection behaviour at selected locations as depicted in Figure 6(a). Also, the contribution of secondary systems to the mitigation of sidesway collapse in Moment-Resisting Frames (MRF) was studied by means of bi-linear oscillators with degrading characteristics (Lignos and Krawinkler, 2011) coupled with PR secondary frame models.

The suite of 20 ground motion records used by Vamvatsikos and Cornell (2002) to analyse mid-rise buildings is used in this study to perform the IDA analyses. Peak ground acceleration (PGA) is considered here as the IM due to the consistent and relatively low values of log-normal standard deviation on PGA observed in this particular suite of records for periods up to 1.6 seconds (Mander *et al.*, 2007). A number of structures with different stability coefficients, θ , which represents the ratio between secondary and primary actions, are considered (i.e. $\theta = 0.1, 0.2$ and 0.3). Similarly, different levels of secondary system stiffness contributions ranging from 0% to 25% of the initial stiffness of the whole structure are studied. Each structure is characterized by its normalized slenderness value ($\bar{\lambda}$) defined as: $\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}}$, where A is the brace area, f_y is its yield strength and N_{cr} is the Euler buckling load. In total, more than 33000 analyses were performed. The comparisons in terms of mean IM at collapse are summarised in Figure 6 and discussed below.

Figures 6(b), 6(d) and 6(f) present the comparisons of median IM at collapse for structures with different levels of secondary frame contribution (IM_{CP+S}) normalized by the collapse capacities of the corresponding original systems with nominally-pinned secondary frames (IM_{CP}) whereas Figures 6(c), 6(e) and 6(g) depict their associated Coefficients of Variation (COV). The secondary system contribution is expressed in terms of the ratio between the initial stiffness corresponding to the whole building (K_{P+S}) and the stiffness of the primary system alone (K_P). It is apparent from these figures that the greater the secondary PR contribution the higher the collapse capacity for both CBF and bi-linear (MRF) structures. This tendency is observed to hold irrespectively of the brace slenderness in the case of CBF systems. On the other hand, the improvement in collapse capacity is also dependent on the stability coefficient θ ,

with higher values of PR contributions needed for systems exhibiting larger stability coefficients in order to achieve comparable collapse capacities. In general, up to a three-fold increase in median collapse intensities (IM_c) can be expected if the secondary PR system is designed to provide adequate stiffness and satisfy a maximum stability coefficient (e.g. systems with PR stiffness contribution $\geq 20\%$ and $\theta \leq 0.2$). It is also shown in Figures 6(b), 6(d) and 6(f) that the results for different values of brace slenderness ($\bar{\lambda}$) are practically coincident when $\theta = 0.2$ or $\theta = 0.3$. However, for $\theta = 0.1$ a larger difference in the mean collapse capacity ratio depending on $\bar{\lambda}$ is observed. This suggest that for systems with lower second-order effects, the hysteretic response of the primary system influences greatly the collapse response of the structures. However, as the second-order effects become more significant, the collapse response is increasingly governed by P-delta effects and the ultimate capacity is less sensitive to differences in hysteretic behaviour. Also, for low second order levels (i.e. $\theta = 0.1$), bi-linear (MRF) structures show smaller improvements in collapse capacities than similar CBF frames (Figure 6(b)) along the full range of secondary frame contributions studied herein. This can be attributed to the higher collapse capacities of bi-linear systems compared with the more limited energy dissipation capabilities associated with the pinching response typical of CBF frames. Similarly, the pinched hysteresis of CBF frames seems to be related to higher variability in their collapse capacity estimation as indicated in Figures 6(c), 6(e) and 6(g) where a more consistent coefficient of variation (COV in the order of 0.3 - 0.6) is observed for MRF systems.

3.3 Mitigation of progressive collapse due to sudden element removal

Figure 7 summarizes the numerical evaluation of the contribution of secondary systems to the mitigation of progressive collapse due to sudden element removal. The structural model presented in Figure 7(a) was employed to assess the influence of different connection geometries and strength levels on the vertical collapse capacity of building sub-assemblages including both CBF as well as MRF structures. UB 406x178x74 beams and SHS 400x16 columns are employed for the primary elements whereas different sections are used for the braces and secondary beam elements. As before, these models were developed in the Opensees framework (McKenna, 1997) by means of inelastic displacement-based fibre elements including geometric and material non-linearities. An initial out-of-straightness in the order of $1/300$ was employed in all braces and columns. Importantly, a full multi-spring mechanical model was employed for the connections as depicted in Figure 7(a). The axial restraint provided by adjacent bays was taken into account by including an additional axial spring as defined by Vlassis *et al.* (2008) and presented in Figure 7(a). Different connection strengths were analysed as well as three secondary system aspect ratios (i.e. h/L values of 0.06, 0.1 and 0.17 where h is the connection depth and L the beam length). Web cleat, top and seat as well as top, seat and web angle connections of varying flexural (and axial) strengths were employed. The secondary frame connection strength levels (M_{uPR}) were normalized by the corresponding beam plastic capacity (M_{pB}). Figures 7(b), 7(c) and 7(d) present the evolution of load factors at collapse as a function of $\bar{\lambda}$, h/L and M_{uPR}/M_{pB} . The collapse load factors of the building (Λ_{cP+S}) are normalized against the corresponding collapse factors for a similar structure incorporating nominally pinned secondary connections (Λ_{cP}) realised by web-cleat joints with minimum tying force, T, capacities

(i.e. $T = 75kN$ Byfield and Paramasivam, 2007, ODPM, 2004) and negligible rotational resistance. Two element removal scenarios were considered. The first scenario (for which results are summarized in Figure 7(b)) involves the loss of all non-horizontal resisting elements coinciding at the middle joint (i.e. two braces and the column) and is relevant to both CBF and MRF configurations subjected to intermediate column removal. The second scenario considers the loss of all but one brace and is reported in Figure 7(d). Three slenderness values for the braces were evaluated (i.e. $\bar{\lambda} = 0.9, 1.7$ and 2.1). IPDA were performed for increasing levels of vertical load applied instantaneously (i.e. within 0.1 second).

It can be appreciated from Figure 7(b) that the progressive collapse capacity is greatly affected by both the aspect ratio (h/L) and secondary frame connection strength (M_{uPR}/M_{pB}). It is important to note that in reality there is a direct relationship between the secondary connection flexural and axial strengths. Peak values of collapse capacity ($\Lambda_{cP+S}/\Lambda_{cP}$) are obtained for secondary connections with ultimate flexural strengths in the order of 20-30% of the beam plastic capacity (corresponding to connections with pure axial yield capacities in the order of 4-5% of the axial yield strength of the jointed beam) after which steady decrements of the load factor at collapse are observed. It is also evident from Figure 7(b) that more significant enhancements in the progressive collapse capacity are obtained for h/L ratios of 0.1 whereas both larger ($h/L = 0.17$) or smaller ($h/L = 0.06$) aspect ratios produce limited improvements irrespective of the level of secondary system contribution. This is attributed to the higher rotational demands imposed on connections adjacent to shorter beams in the case of $h/L = 0.17$ or the higher moments transmitted to the connection by longer beams with $h/L = 0.06$ for similar levels of vertical displacements and load factors. It is also important to note that, due to the above mentioned ductility considerations, secondary connections with flexural capacities larger than 0.45-0.50 times the beam flexural plastic capacity (equivalent to connection axial yield strengths of 0.07-0.09 times the corresponding beam axial yield capacity) may lead to detrimental effects (i.e. $\Lambda_{cP+S}/\Lambda_{cP} \leq 1$) for beams with $h/L = 0.06$. Similarly, strengthening the secondary connections may not produce any benefit for structures with shorter span ratios (i.e. for $h/L = 0.17$). This reduction in the collapse capacity for high-strength secondary connections is related to the provision of stiffer PR details that tend to transfer higher forces to the bolts and induce more brittle failure modes. This can be further appreciated from Figure 7(c) where the response of structures with and without consideration of bolt fracture is presented. Grade 10.9 bolts ranging from M12 to M24 were employed for the secondary connections with a maximum number of 6 bolt-rows considered depending on the beam height. It is clear from Figure 7(c) that the reduction in ductility capacities associated with $M_{uPR}/M_{pB} > 0.5$ lead to a consistent decrease in the collapse resistance of the overall structure. This decrease is related to the higher prying forces induced by the stiffer details required for stronger connections that favour the accumulation of greater forces and deformations within the bolts hence inducing less ductile failure modes.

The element removal case when a single bracing component is assumed to remain intact is depicted in Figure 7(d). In this case, the progressive collapse response of the structure is also a function of the slenderness of the surviving brace. However, lower levels of improvement are brought about by the contribution of secondary systems (e.g. a maximum $\Lambda_{cP+S}/\Lambda_{cP} = 1.5$ for $\bar{\lambda} = 1.0$ in Figure 7(d) compared with $\Lambda_{cP+S}/\Lambda_{cP} = 1.8$ for $h/L = 0.10$ in Figure 7(b)). This is attributed to the already large loads required to

buckle the brace in the original and damaged structures which, once exceeded, are able to generate significant demands on the secondary connections.

3.4 Mitigation of progressive collapse due to localized fires

Figure 8 summarizes the numerical evaluation of the contribution of secondary systems to the mitigation of progressive collapse due to a localized fire. The multi-linear approach described before was employed herein to model the connections. Besides, the idealised Temperature-Independent Approach (TIA) proposed by Fang *et al.* (2011) which avoids complex heat transfer and detailed temperature distribution analysis was adopted. This approach offers a simplified analytical framework which is able to deal with fire conditions, dynamic effects and ductility considerations particularly suited for the extensive parametric analysis carried out as part of the present study. Nevertheless, it should be noted that the simplifications adopted in the TIA can lead to over-conservative predictions for cases in which connection punching shear failure is the main cause of collapse (Fang *et al.*, 2011). Likewise, due to the parametric nature of the analysis presented in this section and in consistency with the event independent framework presented above, a uniform temperature was assumed along the fire-affected area characterized by the radius (r) in Figure 8(a). To this end, the column, beam segments and corresponding connections enclosed within the fire-affected area were assumed to lose all their resistance in accordance with TIA (Fang *et al.*, 2011). UB 406x178x74 beams and SHS 400x16 columns were employed for the primary elements. The models were developed in the Openses framework (McKenna, 1997) by means of inelastic displacement-based fibre elements including geometric and material non-linearities whereas a full multi-spring mechanical model was employed for the connections (Figure 8(a)). Web cleat, top and seat as well as top, seat and web angle connections of varying flexural (and axial) strengths were employed. A number of fire scenarios were studied by varying the extent (r) of the fire-affected zone and the results are summarized in Figures 8(b), 8(c) and 8(d). These figures present the variation of load factors at collapse (Λ_c) normalized by the corresponding collapse load factors of the original undamaged structure ($\Lambda_{cr/L=0}$). Results of collapse capacities are presented as a function of different secondary system aspect ratios (i.e. $h/L = 0.06, 0.1$ and 0.17) and strength levels (M_{uPR}/M_{pB} between 0.1 and 0.9).

Figure 8(b) presents the variation of normalized load factor ($\Lambda_c/\Lambda_{cr/L=0}$) as obtained from IPDA against the size of the fire-affected area (r/L). The results presented in Figure 8(b) correspond to secondary connection flexural strengths of $M_{uPR}/M_{pB} = 0.1$ which, for the structure under consideration, represent roughly the provision of minimum tying force requirements (i.e. $T = 75kN$ Byfield and Paramasivam, 2007, ODPM, 2004). It can be seen from Figure 8(b) that a collapse capacity plateau is reached after 15% - 30% of the beam lengths within the fire-affected column loses its load carrying capacity depending on h/L . This is due to the fact that the connections immediately adjacent to the fire-affected column, which contribute significantly to the vertical resistance, are affected almost simultaneously. The structure is therefore rendered relatively insensitive to further beam damage as long as the beams are able to maintain enough tension capacity to avoid fracture. Also, a steeper decrease in collapse capacity is observed for deeper beams due to the lower ductility of the connections associated with them. Similarly, a residual capacity in the order of 62% to 68% of the total resistance of the

structure is maintained for $r/L > 0.3$ depending on the beam aspect ratio (h/L).

The influence of the secondary connection strength can be further discussed with reference to Figure 8(c) which presents the variation of load factors at collapse for connection flexural strength ratios, M_{uPR}/M_{pB} , of 0.1, 0.5 and 0.9 (roughly equivalent to ratios between connection and beam pure axial yield capacities of 0.05, 0.10 and 0.15). A noticeable improvement in the collapse capacity of the sub-assembly is evident when the connection strength is enhanced from $M_{uPR}/M_{pB} = 0.1$ to $M_{uPR}/M_{pB} = 0.5$. Stronger secondary connections (i.e. $M_{uPR}/M_{pB} = 0.9$) do not appear to be effective in further improving the overall vertical (progressive) collapse resistance of the sub-assembly and may even become detrimental when larger fire-affected areas are involved as is the case for $M_{uPR}/M_{pB} = 0.9$ in comparison with $M_{uPR}/M_{pB} = 0.5$ for $h/L > 0.25$. This is consistent with the findings discussed above for sudden element removal scenarios where the stiffer connections components associated with higher joint resistances limit the ductility capacity of the connection by inducing more brittle failure modes in the connection (e.g. bolt fracture).

If a gradual (rather than a sudden) loss of strength in the structural elements is envisaged, a Static Push-down Analysis (SPA) can be performed as opposed to a full dynamic IDPA analysis. In those cases, a gradual loss of vertical carrying capacity is assumed and all dynamic effects are disregarded. A direct relationship can then be obtained between a monotonically increasing vertical load and the corresponding vertical displacement at a critical location. To this end, Fang *et al.* (2012) showed that for the case of columns directly affected by localized fire, the structural response lies in-between the extreme theoretical cases of sudden column removal and a full static behaviour. A comparison between the results of static (SPA) and dynamic (IDPA) push-down analysis is presented in Figure 8(d) for $h/L = 0.10$ and $M_{uPR}/M_{pB} = 0.10$. It can be appreciated from this figure that the estimations of vertical collapse capacities obtained from idealized static analysis are consistently larger than those obtained by means of dynamic models. A nearly constant 10% difference is noticeable along the full range of fire sizes studied. The consistency of these results highlights the applicability of static procedures for preliminary design calculations.

3.5 Design considerations

The previous section has highlighted the influence of the stiffness, strength and ductility of secondary systems on the collapse resistance of sub-structures representative of steel buildings. On this basis, this section aims to offer some discussion on directions for the dimensioning of secondary frames and in particular secondary-frame connections. The main objective in designing the stiffness and strength contributions of secondary frames against sidesway collapse becomes apparent if the response of primary and secondary frames is assumed as consisting of two lateral resisting systems acting in parallel. Consequently, the secondary system design can be targeted to modify the post-yield stiffness and ultimate capacity of the overall structure (Vargas and Bruenau, 2006, Málaga-Chuquitaype and Elghazouli, 2010a). To this end, the secondary system lateral stiffness K_S can be expressed as:

$$K_S \geq K_P(h_{P+S} - h_P)/(1 - h_{P+S}) \quad (8)$$

where K_P is the lateral stiffness of the primary system, and h_P and h_{P+S} are the post-elastic hardening ratios for the primary system and the whole building, respectively. By assuming that K_P and h_P are known (or can be estimated), and given a target post-elastic hardening ratio h_{P+S} that can be established as the value of hardening stiffness that will offset secondary effects (e.g. $h_{P+S} = 0.1$ for $\theta = 0.1$ from Figure 6), the secondary system lateral stiffness can be determined from Equation 8.

Once the target post-elastic stiffness of the secondary system is determined, the stiffness required from the secondary system PR-connections (S_θ) can be determined by assuming that K_S is the sum of the stiffness contributions associated with the columns (k_c), beams (k_b) and connections (k_θ) as:

$$S_\theta = h^2 \frac{l_c}{l} k_\theta = h^2 \frac{l_c}{l} \frac{K_s k_c k_b}{k_b k_c - (k_b + k_c) K_s} \quad (9)$$

where l is the bay length, l_c is the distance between the bay mid-span and the column face, and k_b and k_c are the stiffnesses of the beam and column sub-assemblages.

Also, based on the results summarized in Figures 7 and 8, it seems necessary to impose a limit on the maximum strength of the secondary connections in order to prevent detrimental effects caused by the reduction in joint ductility associated with stronger connections against vertical (progressive) collapse. It has been noted with reference to Figure 7 that a maximum enhancement in vertical (progressive) collapse capacity is obtained for M_{uPR}/M_{pB} ratios in the order of 0.2 when sudden element removal actions were considered. Similarly, it was observed from Figures 7 and 8 corresponding to sudden column removal and localized fire scenarios, that M_{uPR}/M_{pB} values greater than 0.5 do not result in significant improvement in the collapse capacity, and can even be detrimental. In general, a combined assessment of the interdependencies between connection stiffness, capacity and ductility should be carried out in light of the levels of risk associated with progressive or sidesway collapse as illustrated in the case studies discussed below. In this context, the findings of this section, together with the collapse assessment approach described in Section 2, provide the necessary framework for an integrated analysis of possible design options. The next section applies this collapse assessment framework to the evaluation of total collapse risks in typical examples of CBF and MRF subjected to combined seismic and blast hazards, and combined blast and fire hazards, respectively.

4. Case studies on multi-storey structures

4.1 Six-storey CBF under seismic and blast hazards

The six-storey CBF building presented in Figure 9(a) was analysed here under blast and seismic hazard scenarios. This building has external CBF frames acting as a primary lateral resisting system and internal PR gravity frames. Two designs were considered: (i) firstly, the internal frames were assumed to resist gravity loads only and web-cleat nominally pinned connections were designed for the secondary system with due consideration of the minimum robustness requirements (i.e. connection tying force of 75kN); (ii) a second (alternative) design was performed following the considerations outlined above with a target post-elastic stiffness for the overall structure of 10%

incorporating semi-rigid secondary connections. Care was also placed in maintaining the flexural strengths below the $M_{uPR}/M_{pB} \approx 0.5$ range which marks the onset of potential detrimental effects caused by the ductility reductions observed for stronger connection configurations, as explained above. The details of both designs are summarized in Table 1.

A detailed model was constructed in Opensees for both designs with due account of material and geometric non-linearities. Columns were modelled by means of elastic elements with concentrated plasticity at the extremes including degradation effects (Lignos *et al.*, 2009). Braces were discretized in a number of fibre-based elements accounting for fatigue fracture (Uriz and Mahin, 2008) with initial imperfections at mid length in the order of 1/300. The connections in the secondary system were represented by rotational springs when seismic action was considered whereas multi-spring component based models were employed in the case of progressive collapse analyses.

The seismic risk assessment performed herein is based on the results of IDA using the same suite of 20 records employed for the sub-assembly analyses of the previous section (Vamvatsikos and Cornell, 2002). The spectral acceleration at the initial period of the building is used as the intensity measure (IM). Figure 9(b) presents the comparison of fragility curves for the building under study. It can be observed that an adequate design of the secondary frames improves its median collapse capacity by 40% (from 0.52g to 0.72g).

Regarding the blast scenarios, all storeys were subject to the potential blast effects of a backpack bomb carrying between 15 and 35 kg of equivalent TNT with equal probability. To this end, a set of 600 realizations were generated by means of Monte Carlo simulations. Based on the distance from the charge and the amount of explosives corresponding to each realization, a triangular blast impulse was calculated. This blast impulse load was applied to the individual elements of the structure and a simplified local dynamic analysis was performed in order to verify whether the corresponding element survived the explosion by employing the closed-form solutions developed by Asprone *et al.* (2010). After identifying and removing the damaged elements, an IDPA was performed and the corresponding collapse capacity was determined. Figure 9(c) presents the cumulative distribution function of the load factor Λ such that $\Lambda \leq \Lambda_c$, where Λ_c corresponds to the load multiplier that would cause collapse in the structure. If a load factor equal to 1 is assumed (i.e. implying that the building was subjected to dead loads plus 30% of the nominal live loads at the moment of the element removal), a 60% reduction in the collapse probability can be identified for the improved secondary system design as compared with the conventional design (i.e. $P[\Lambda \leq \Lambda_c | Blast] = 0.21$ for the conventional design versus $P[\Lambda \leq \Lambda_c | Blast] = 0.08$ for the alternative design). The step at $\Lambda_c \approx 6$ observed in the curves of Figure 9(c) represents the limit response at which even if no element is removed, the buildings would collapse under the instantaneous application of 6 times the normal operating loads.

A simple hazard recurrence relationship, between the earthquake intensity (IM) and the annual frequency (f_a) is adopted, as follows:

$$f_a(IM) = a(IM)^{-b} \quad (10)$$

where a and b are empirical parameters assumed here as 1.00 and 3.00, respectively (Mander *et al.*, 2007). Consequently, by integrating the hazard with the fragility curve

presented in Figure 9(b) a mean seismic collapse risk of $1.2\text{E-}4$ is calculated for the conventional design whereas a $5.0\text{E-}5$ risk is evaluated for the alternative secondary system design. Similarly, if the structure under study corresponds to a strategic building, an annual rate that a significant blast event takes place of $\nu_E = 1.0\text{E-}4$ can be assumed (Ellingwood, 2006). By multiplying ν_E with the mean probability of collapse $P[\Lambda \leq \Lambda_c | \text{Blast}]$ for $\Lambda = 1$ (Figure 9(c)) a mean collapse risk due to blast of $2.1\text{E-}5$ is calculated for the conventional design whereas a $8.0\text{E-}6$ risk is evaluated for the alternative secondary system design. It can be seen from these values that an adequate consideration and dimensioning of secondary PR frames can reduce the total annual rate of collapse by 60% from $1.4\text{E-}4/\text{year}$ to $5.8\text{E-}5/\text{year}$. These total annual rates of collapse should be compared with the *de minimis* threshold (Equation 3) in order to establish the adequacy of the design. Additionally, it can be concluded that for this particular case, the blast hazard contributes 15% to the total probability of collapse of the building, which will be reduced to less than 1% if the building under consideration was a non-strategic facility. This evaluation would justify a stiffness-based only design of secondary connections with the possibility of employing connections with capacities larger than the $M_{uPR}/M_{pB} = 0.5$ threshold identified in the previous section, for non-strategic structures where the main contributor to the annual rate of collapse is the seismic hazard.

4.2 Four-storey MRF under blast and fire hazards

Figure 10(a) presents a four-storey MRF building considered herein to illustrate the contribution of secondary (gravity) frames to the mitigation of total collapse probabilities under blast and fire hazards at its bottom storey. The MRF under consideration is composed of external primary MRF frames and internal PR gravity frames and incorporates a car-park at its bottom storey. As, before, two designs were performed: (i) firstly, the connections in all internal frames were designed to provide the minimum robustness requirements specified by a tying force of 75kN whereas fully-rigid connections were employed in the external MRF frames; (ii) a second (alternative) design was performed on which secondary-frame connections were provided with strength in the order of $M_{uPR}/M_{pB} \approx 0.3$ in order to maximise their positive contributions as discussed above. Table 1 summarizes the element sections employed in both designs. It should be noted that in this case, unlike the previous CBF case study where stiffer column sections were required in the upper storeys (Section 4.1), the target stiffness and strength was achieved through an improved design of the connections alone and no changes were implemented in the column or beam sections. To this end, the alternative design incorporated top, seat and web-angle connections with 6 M20 bolt-rows as opposed to the web cleat detail with 2 bolt-rows employed to satisfy the minimum tying force requirements in the original design. A detailed model was constructed in Opensees with columns and beams modelled by means of fibre-based elements whereas component-based models were employed for the PR connections.

The blast hazard in the bottom storey of the MRF was characterized by a car-bomb carrying between 30 kg and 100 kg of equivalent TNT. 100 realizations were generated by means of Monte Carlo simulations and the closed form expressions for the determination of the elements to be removed proposed by Asprone *et al.* (2010) were used as before. The results of the IDPA analysis are summarized in Figure 10(b) in terms of the cumulative distribution function of the load factor Λ , such that $\Lambda \leq \Lambda_c$, where Λ_c

corresponds to the load multiplier that would cause collapse in the structure. It can be observed from Figure 10(b) that no collapse case was encountered for a load factor (Λ) equal to 1 in both designs. However, a notable difference in the cumulative probabilities of collapse is evident with the alternative design incorporating PR connections, reducing significantly the conditional probability of collapse for the whole $1 < \Lambda < 4$ range. As before, the building would collapse at $\Lambda_c \geq 5.5$ for the conventionally designed MRF and at $\Lambda_c \geq 5.7$ for the alternative design even if no element is removed.

With regards to localized fire, only the vulnerability of the building to single vehicle fires in the car-park at the bottom storey is considered and the probability of fire spreading to neighbouring vehicles is assumed to be negligible (Li and Spearpoint, 2007). The location and extent of the fire affected areas were determined on the basis of randomly generated fire durations and in accordance with codified Heat Release Rate curves (CEN, 2005b, CEN, 2002b). The simplified fire model proposed by Fang *et al.* (2013) was employed to verify the critical loss of load carrying capacity in the temperature domain instead of the conventional time domain. To this end, the critical temperature was defined as the temperature at which failure is expected to occur in a structural steel element for a uniform temperature distribution (CEN, 2005b). Following the removal of the fire-affected element segments that reached the critical temperature, IDPA analyses were performed and the load factor at collapse was determined for each of the 100 scenarios considered.

Figure 10(c) summarizes the results of the localized fire analysis in the MRF under consideration. From this figure it can be appreciated that if a load factor equal to 1 is assumed, no significant difference in the collapse probability exists between the conventional design and the design incorporating improved PR secondary connections. However, a notable difference in the cumulative probabilities of collapse is evident if a slightly more conservative value of $\Lambda = 1.5$ is assumed. In this case, the probability of collapse of the MRF designed to minimum tying force requirements nearly doubles the collapse probability corresponding to the MRF incorporating stronger PR secondary connections. If a fire frequency of $1.7\text{E-}7$ is assumed (Li and Spearpoint, 2007), a mean collapse risk due to localized fire in the bottom storey of $1.4\text{E-}7$ is calculated for the conventional design whereas a $6.0\text{E-}8$ risk is evaluated for the alternative secondary system design. It can be seen from these results that a 40% reduction in the collapse probability under a localized fire is achievable by an adequate consideration of the secondary (gravity) frame contribution. Also important is the realization that the MRF structure considered is significantly more vulnerable to collapse due to localized fire than to blast action. This may in part be related to the fact that the connection under the fire affected region is prone to significant loss of strength whereas only column damage was considered during the blast analysis.

5. Conclusions

This paper evaluated the contribution of secondary systems to the mitigation of building collapse due to extreme loading conditions arising from multiple hazards. Two main collapse modes were assumed to encompass most building collapse cases: i) sidesway collapse due to seismic action and ii) vertical progressive collapse represented by the

scenario-independent element removal approach.

An assessment of the influence of partially-restrained gravity frames on the sidesway collapse capacity of structures by means of generalized structural sub-assemblages was undertaken. The influence of connection stiffness, capacity and aspect ratio (connection depth to beam span ratio) were discussed. The results suggest that adequately proportioned secondary systems can improve the seismic collapse capacity of steel buildings regardless of the brace slenderness. However, a greater PR connection stiffness would be required for structures with higher second order effects (represented by larger values of the stability coefficient). This study also revealed the importance of controlling the connection stiffness and strength which stand in inverse relationship to the connection ductility capacities, with a limit on the connection to beam flexural capacity ratio (M_{uPR}/M_{pB}) of 0.5 providing reasonable results for the range of sub-assemblages and multi-storey buildings considered here. To this end, more significant enhancements in the vertical (progressive) collapse capacity were obtained for connection depth to beam length ratios (h/L) ratios of 0.1 whereas both larger ($h/L = 0.17$) or smaller ($h/L = 0.06$) aspect ratios produce limited improvements irrespective of the level of secondary system contribution. In general, this paper has shown that appropriate detailing and deployment of secondary systems, which are normally designed for gravity loads or minimum tying forces, can lead to considerable benefits in terms of the reduction of collapse probabilities in structures subjected to multiple hazards and that a probabilistic comparison of different design alternatives is within the reach of practical assessment and design.

References

- [1] D Asprone, F. Jalayer, and G. Manfredi. (2010). Proposal of a probabilistic model for multi-hazard risk assessment of structures in seismic zones subjected to blast for the limit state of collapse. *Structural Safety*, 32:25–34.
- [2] M. Byfield and S. Paramasivam. (2007). Catenary action in steel-framed buildings. *Structures and Buildings*, 160(5):247–257.
- [3] Comité Européen de Normalisation. (2002a). EN 1990, Eurocode - basis of structural design.
- [4] Comité Européen de Normalisation. (2002b). EN 1991, Eurocode 1: Actions on structures - Part 1-2: General actions - Actions on structures exposed to fire.
- [5] Comité Européen de Normalisation. (2005a). EN 1998-3, Eurocode 8: Design of structures for earthquake resistance, Part 3: Assessment and retrofit of buildings.
- [6] Comité Européen de Normalisation. (2005b). EN 1993-1-2, Eurocode 3: Design of steel structures - Part 1-2: General rules - Structural fire design.
- [7] M. Dicleli and A. Mehta. (2007). Simulation of inelastic cyclic buckling behaviour of steel box sections. *Computers and Structures*, 85:446–457.
- [8] Department of Defense. (2005). Design of buildings to resist progressive collapse. Unified Facilities Criteria (UFC), US.
- [9] M. D’Aniello, G. Ambrosino, F. Prtioli, and R. Landolfo. (2013). Modelling aspects of the seismic response of steel concentric braced frames. *Steel and Composite Structures*, 15(5):539–566.
- [10] A.Y. Elghazouli, C. Málaga-Chuquitaype, J.M. Castro, and A.H. Orton. (2009).

- Experimental monotonic and cyclic behaviour of blind-bolted angle connections. *Engineering Structures*, 31(11):2540–2553.
- [11] B. Ellingwood and E.E. Leyendecker. (1978). Approaches for design against progressive collapse. *Journal of the Structural Division, American Society of Civil Engineering*, 104(3):413–423.
- [12] B. Ellingwood. (2006). Mitigating risk from abnormal loads and progressive collapse. *Journal of Performance of Constructed Facilities, American Society of Civil Engineering*, 20:315–323.
- [13] C. Fang, B. Izzuddin, A.Y. Elghazouli, and D. Nethercot. (2011). Robustness of steel-composite building structures subject to localised fire. *Fire Safety Journal*, 46:348–363.
- [14] C. Fang, B. Izzuddin, R. Obiala, A.Y. Elghazouli, and D. Nethercot. (2012). Robustness of multi-storey car parks under vehicle fire. *Journal of Constructional Steel Research*, 75:72–84.
- [15] C. Fang, B. Izzuddin, A.Y. Elghazouli, and D. Nethercot. (2013). Robustness of multi-storey car parks under local fire - Towards practical design recommendations. *Journal of Constructional Steel Research*, 90:193–208.
- [16] General Services Administration. (2003). Progressive collapse analysis and design guidance for new federal office buildings and major modernization projects, US.
- [17] P. Hsiao, D.E. Lehmann, and C.W. Roeder. (2013). Evaluation of the response modification coefficient and collapse potential of special concentrically braced frames. *Earthquake Engineering and Structural Dynamics*, 42:1547–1564.
- [18] L.F. Ibarra and H. Krawinkler. (2005). Global collapse of frame structures under seismic excitation. TR -152. Technical report, The John A. Blume Earthquake Engineering Center, Department of Civil and Environmental Engineering, Stanford University, US.
- [19] B. Izzuddin, A.G. Vlassis, A.Y. Elghazouli, and D. Nethercot. (2008). Progressive collapse of multi-storey buildings due to sudden column loss - Part I: Simplified assessment framework. *Engineering Structures*, 30:1308–1318.
- [20] S. Kiggins and C.M. Uang. (2006). Reducing residual drift of buckling restrained braced frames as dual systems. *Engineering Structures*, 28(11):1525–1532.
- [21] T. Kim, J. Kim, and J. Park. (2009). Investigation of progressive collapse-resisting capability of steel moment frames using push-down analysis. *Journal of Performance of Constructed Facilities, American Society of Civil Engineering*, 5:327–335.
- [22] Y. Li and M. Spearpoint. (2007). Analysis of vehicle fire statistics in New Zealand parking buildings. *Fire Technology*, 43:93–106.
- [23] Y. Li, A. Ahuja, and E. Padgett. (2012). Review of methods to assess, design for, and mitigate multiple hazards. *Journal of Performance of Constructed Facilities, American Society of Civil Engineering*, 26:104–117.
- [24] D.G. Lignos, H. Krawinkler, and F. Zareian. (2009). Modelling of component deterioration for collapse prediction of steel frames. In *6th International Conference on Behavior of Steel Structures in Seismic Areas, STESSA 2009, Philadelphia, Pennsylvania, US*.
- [25] D. Lignos and H. Krawinkler. (2011). Deterioration modeling of steel components in support of collapse prediction of steel moment frames under earthquake loading. *Journal of Structural Engineering, American Society of Civil Engineering*, 137(11):1291–1302.
- [26] C. Málaga-Chuquitaype and A.Y. Elghazouli. (2010a). Contribution of secondary systems to the seismic performance of steel framed structures. In *14th World*

- Conference on Earthquake Engineering, Ohrid, Macenodia. Paper 235.*
- [27] C. Málaga-Chuquitaype and A.Y. Elghazouli. (2010b) Component-based mechanical models for blind-bolted angle connections. *Engineering Structures*, 32:3048–3067.
 - [28] C. Málaga-Chuquitaype, A.Y. Elghazouli, and Y. Liu. (2014). Response of blind-bolted connections under extreme dynamic actions. In *7th European Conference on Steel and Composite Structures*.
 - [29] L.J. Malvar and J.E. Crawford.(1998). Dynamic increase factors for steel reinforcing bars. Technical report, Twenty-eight department of defence explosives safety board (DDESB).
 - [30] J.B. Mander, R.J. Dhakal, N. Mashiko, and K.M. Solberg. (2007). Incremental dynamic analysis applied to seismic financial risk assessment of bridges. *Engineering Structures*, 29(10):2662–2672.
 - [31] F. McKenna. (1997). *Object Oriented Finite Element Programming: Frameworks for Analysis, Algorithms and Parallel Computing*. PhD thesis, University of California, Berkeley, Berkeley, CA, US. <http://opensees.berkeley.edu>.
 - [32] Office of the Deputy Primer Minister. (2004). The building regulations 2000, Part A, Schedule 1:A3, Disproportionate collapse, London, UK.
 - [33] D. Pettinga, C. Christopolous, S. Pampanin, and M.J.N. Priestley. (2007). Effectiveness of simple approaches in mitigating residual deformations in buildings. *Earthquake Engineering and Structural Dynamics*, 36(12):1765–1785.
 - [34] J.A. Swanson and R. T. Leon. (2001). Stiffness modeling of bolted T-stub connection components. *Journal of Structural Engineering, American Society of Civil Engineering*, 127(5):498–505.
 - [35] P. Uriz and S.A. Mahin. (2008). Towards earthquake-resistant design of concentrically braced steel-frame structures PEER Report 2008/08. Technical report, Pacific Earthquake Engineering Research Center, University of California at Berkeley, California, US.
 - [36] D. Vamvatsikos and C.A. Cornell. (2002). Incremental dynamic analysis. *Earthquake Engineering and Structural Dynamics*, 31(3):491–514.
 - [37] A.G. Vlassis, B. Izzuddin, A.Y. Elghazouli, and D. Nethercot. (2008). Progressive collapse of multi-storey buildings due to sudden column loss - Part II: Application. *Engineering Structures*, 30:1424–1438.
 - [38] R. Vargas and M. Bruneau. (2006). Analytical investigation of the structural fuse concept. Report MCEER-06-004. Technical report, Multidisciplinary Center for Earthquake Engineering Research, State Univ. of New York, Buffalo, N.Y.

Tables

Frame	Primary beams (storey)	Secondary beams (storey)	Primary columns (storey)	Secondary columns (storey)	Braces (storey)
CBF - Secondary frames with nominally pinned connections	IPE 550 (1-2), IPE 500 (3-4), IPE 450 (5), IPE 360 (6)	IPE 450 (1-5), IPE 360 (6)	SHS 400x20 (1), SHS 400x16 (2-3), SHS 350x16 (4-5), SHS 350x10 (6)	SHS 350x16 (1-5), SHS 350x10 (6)	SHS 120x8 (1-2), SHS 120x6.3 (3), SHS 120x5 (4-6)
CBF - Secondary frames with partially-restrained conenctions	IPE 550 (1-2), IPE 500 (3-4), IPE 450 (5), IPE 360 (6)	IPE 450 (1-6)	SHS 400x20 (1), SHS 400x16 (2-3), SHS 350x16 (4-5), SHS 350x10 (6)	SHS 350x16 (1-6)	SHS 120x8 (1-2), SHS 120x6.3 (3), SHS 120x5 (4-6)
MRF - Secondary frames with nominally pinned connections	UB 305x165x54	UB 305x165x54	UC 254x25x73	UC 254x25x73	
MRF - Secondary frames with partially-restrained conenctions	UB 305x165x54	UB 305x165x54	UC 254x25x73	UC 254x25x73	

Table 1. Design details of the steel frames under study

Figures

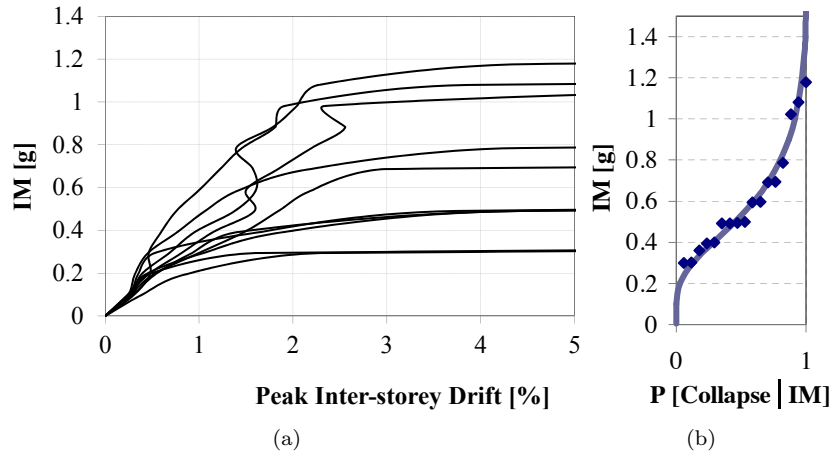


Figure 1. Computation of building sidesway collapse capacity by means of IDA.

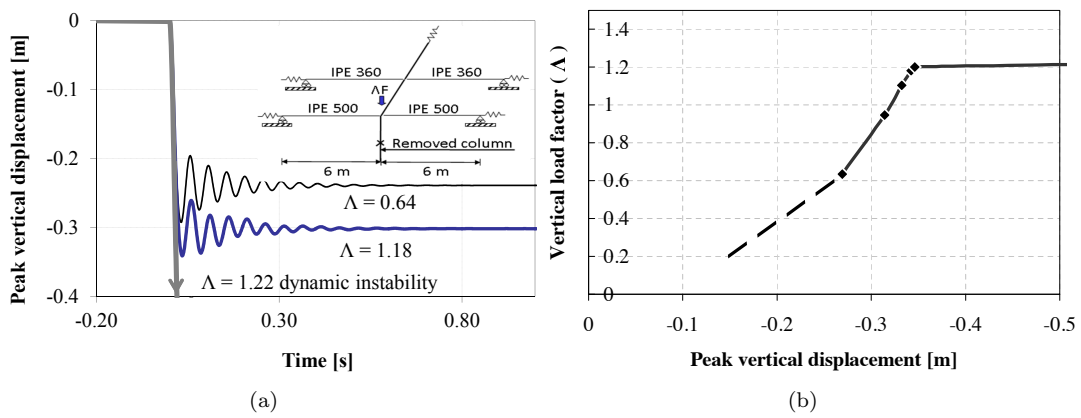


Figure 2. Determination of vertical collapse capacity of a flooring system by means of IDPA.

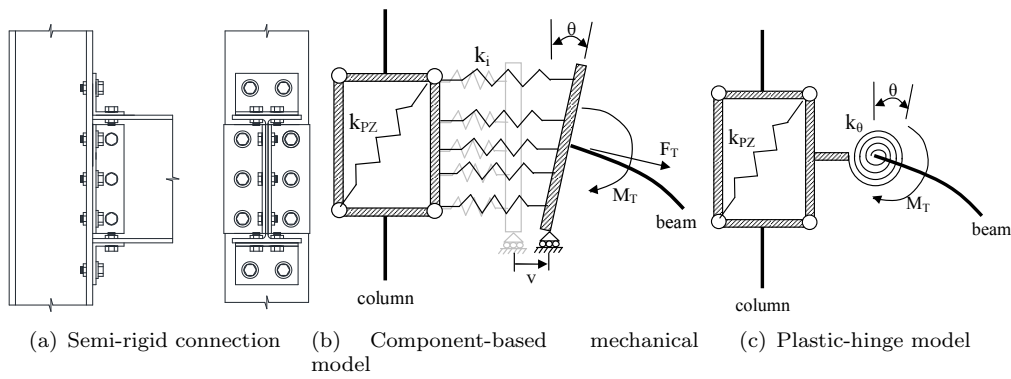
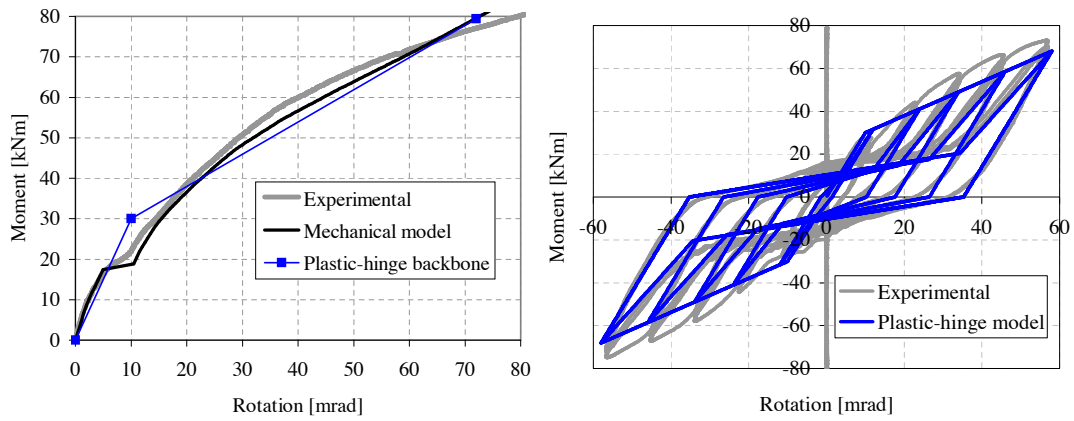
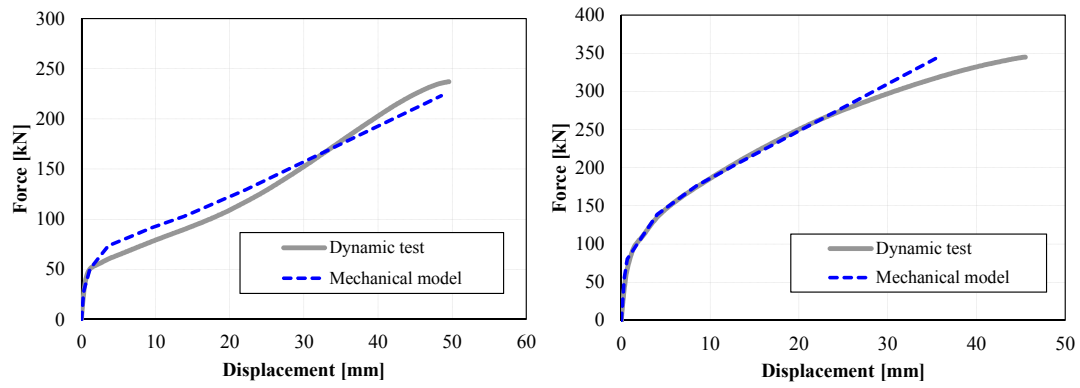


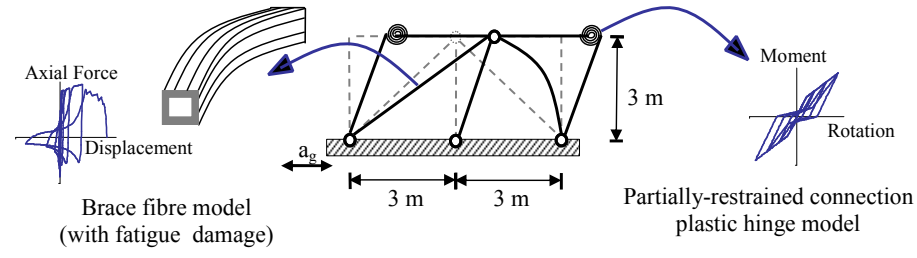
Figure 3. Connection modelling.

Figure 4. Component-base mechanical modelling of PR connections under monotonic and cyclic loading (Specimen C10-G10.9-d40-M (Elghazouli *et al.*, 2009))

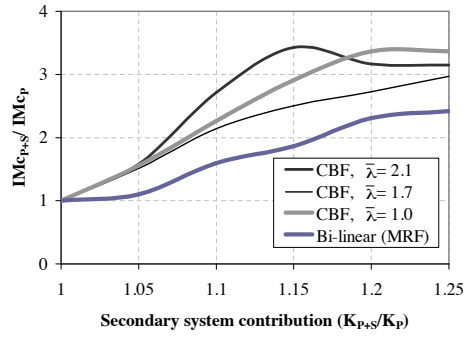


(a) Specimen T1-D (Málaga-Chuquitaype *et al.*, 2014) (b) Specimen T2-D (Málaga-Chuquitaype *et al.*, 2014)

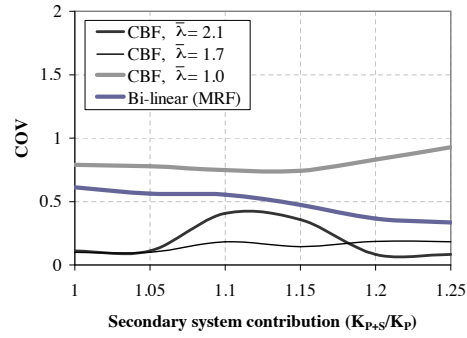
Figure 5. Component-base mechanical modelling of PR connections under dynamic loading with peak strain rates in the order of 20 s^{-1}



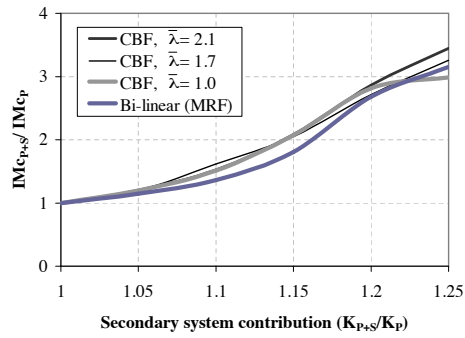
(a) Structural model for sidesway collapse analysis



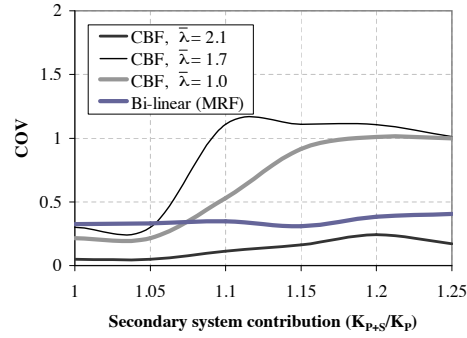
(b) IM ratio, $\theta = 0.1$



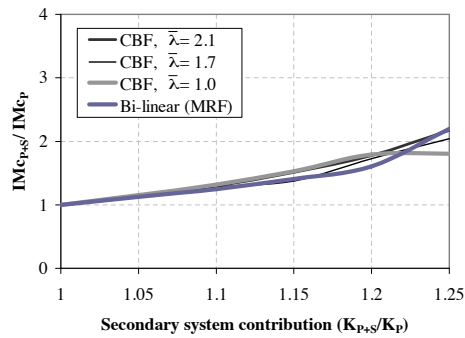
(c) Coefficient of variation, $\theta = 0.1$



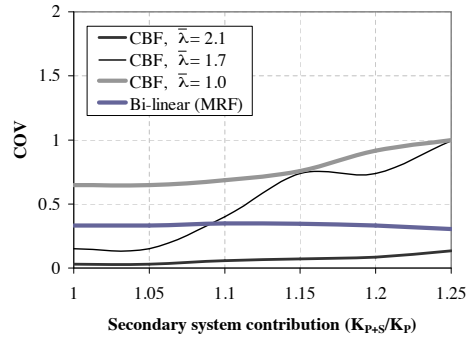
(d) IM ratio, $\theta = 0.2$



(e) Coefficient of variation, $\theta = 0.2$

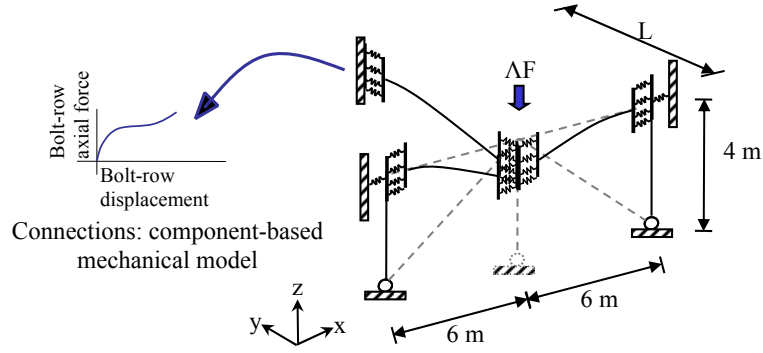


(f) IM ratio, $\theta = 0.3$

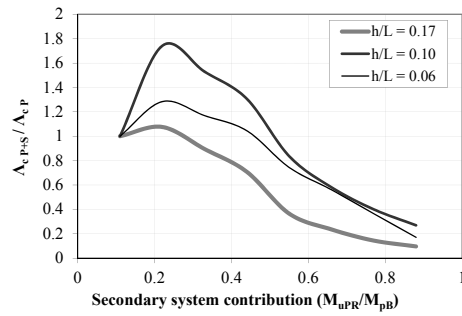


(g) Coefficient of variation, $\theta = 0.3$

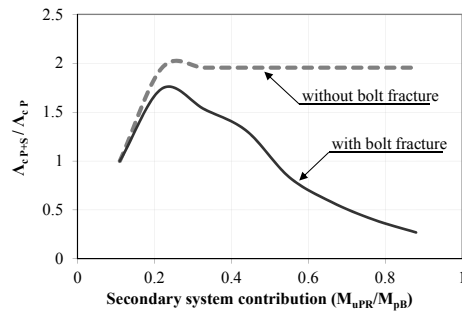
Figure 6. Contribution of secondary systems to the mitigation of sidesway seismic collapse



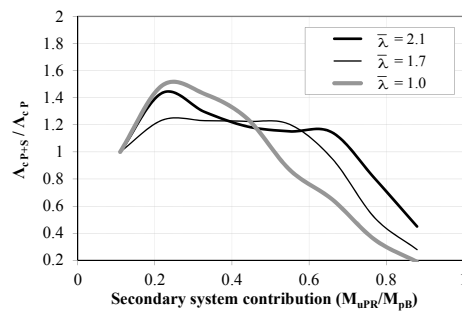
(a) Structural model for sudden element removal collapse analysis



(b) Loss of column and both braces

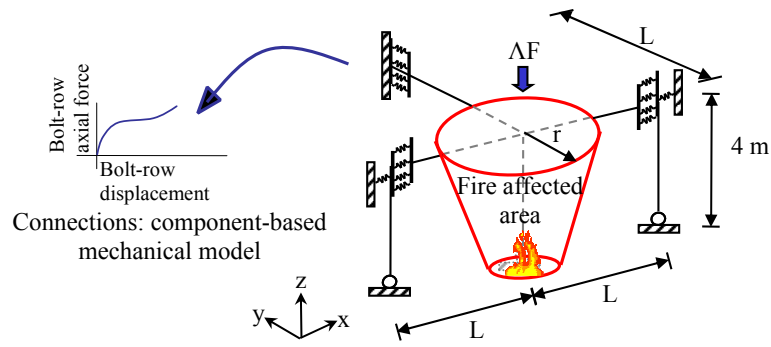


(c) Influence of modelling bolt fracture. Loss of column and both braces. ($h/L = 0.10$)

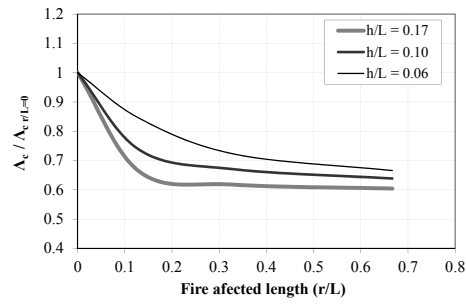


(d) Loss of column and one brace

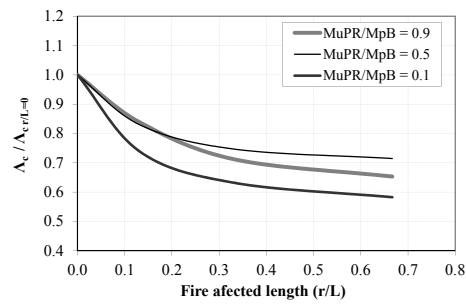
Figure 7. Contribution of secondary systems to the mitigation of progressive collapse under sudden element removal



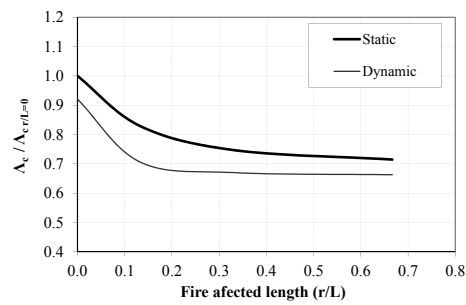
(a) Structural model for localized fire collapse analysis



(b) Variation of fire affected region ($M_{uPR}/M_{pB} = 0.10$)



(c) Influence of secondary frame connection strength ($h/L = 0.10$)



(d) Static versus dynamic push-down analyses ($h/L = 0.10$, $M_{uPR}/M_{pB} = 0.10$)

Figure 8. Contribution of secondary systems to the mitigation of progressive collapse under localized fire

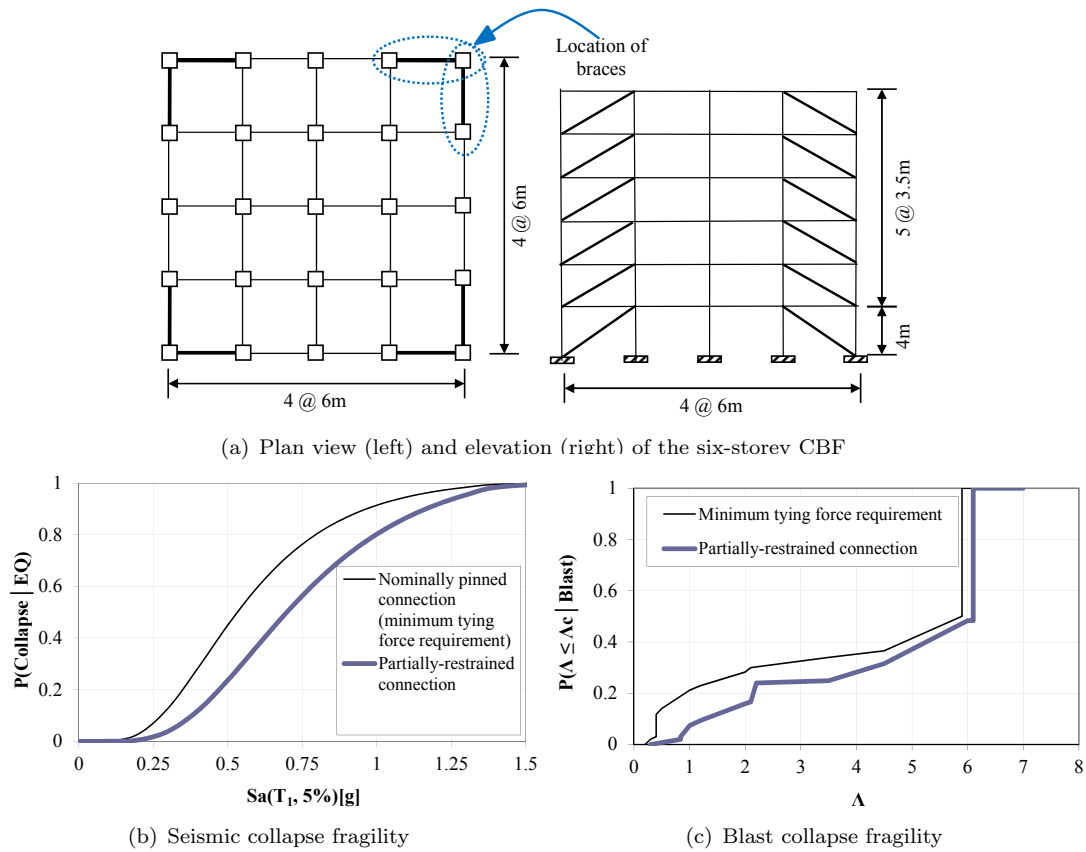


Figure 9. Case study on seismic and blast collapse probability assessment of a six-storey CBF

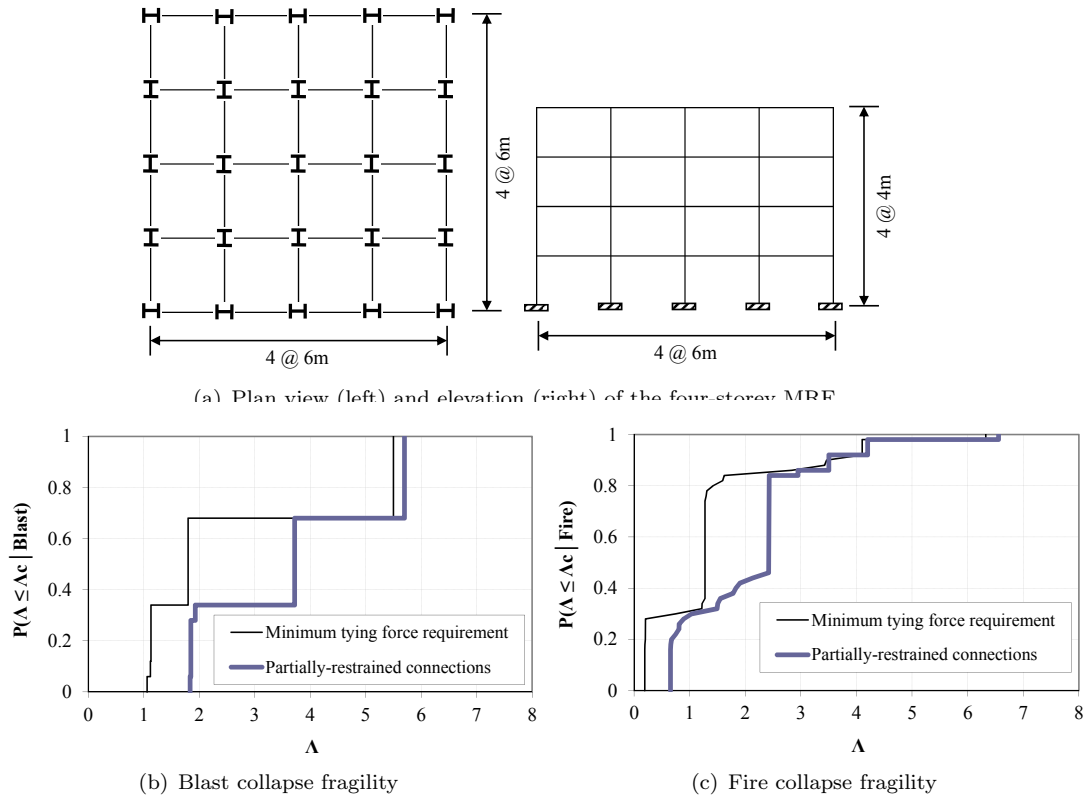


Figure 10. Case study on blast and localized fire collapse probability assessment of a four-storey MRF