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Seismic design of columns in concentrically braced frames with replaceable brace modules

Vahid Mohsenzadeh[1], Lydell Wiebe[2]

[1] Ph.D. Candidate, Department of Civil Engineering, McMaster University - Hamilton, ON, Canada.
[2] Associate Professor, Department of Civil Engineering, McMaster University - Hamilton, ON, Canada.

ABSTRACT

In concentrically braced frames (CBFs), braces are typically connected at beam-column connections through gusset plates. Although this type of connection is widespread in regions of high seismicity, it requires extensive field welding, and it also makes the brace buckle in the out-of-plane direction, which can cause damage to the cladding, potentially endangering people's lives. Moreover, after an earthquake it is difficult to replace the damaged braces, even though this is likely necessary before the building can be returned to service. Recently, a novel brace connection type has been proposed in which bolts are used instead of field welding, the brace will buckle in the in-plane direction, and the brace unit is more easily replaced because all damage is confined to the replaceable brace module. In this connection, the brace module is only connected to the beam and is offset from the column face. Due to this offset, an extra moment will be applied to the columns located in the braced bay, and this must be considered in design. This paper assesses the effect of this eccentricity on the behaviour of a six-storey special concentrically braced frame. The frame is designed according to current seismic design provisions for concentrically braced frames, but using replaceable brace modules instead of more conventional connection detailing. The associated offsets are then included in the model, and nonlinear dynamic analyses using the suite of 44 ground motions from FEMA P695 are conducted. The seismic performance of the frames is discussed in terms of seismic demands on the columns at the DBE (design basis earthquake) and MCE (maximum considered earthquake) levels, as well as the collapse capacity. Based on these results, the additional moment with the replaceable brace module is not significant, but regardless of the connection type, analysis approaches that only take into account the axial demand for designing the columns are not sufficient.

Keywords: Steel structures, Concentrically braced frames, Replaceable brace module, Column seismic demand

INTRODUCTION

Motivation and connection design

One of the most common lateral force resisting systems for steel buildings in seismic zones is a concentrically braced frame (CBF). During an earthquake, the braces are intended to dissipate the energy through buckling, post-buckling and tensile yielding. In current practice, a gusset plate is used to connect the brace to the frame members. To ensure that the brace can buckle in the out-of-plane direction without unintended restrictions or fracture, geometrical limits (linear or elliptical clearance) are considered for the gusset plate, as shown in Figure 1(a-b) [1,2]. According to this detail, field welding is necessary to join the brace to the gusset plate, which is time-consuming and relatively difficult for quality control. Moreover, out-of-plane buckling can cause damage to the adjacent infill walls or cladding, and testing has shown that toe weld fractures can occur at the gusset plate weld due to opening and closing moments on the connections [3]. Complete interface weld fracture prior to brace fracture can negatively affect the ductility and energy dissipation of the system.

To remove all these concerns, an innovative connection detail has been developed based on a replaceable brace module (Figure 1c) [4]. This alternative connection is expected to increase the erection speed by using bolts instead of field welding to connect the brace to the beam, to make the brace buckle in the in-plane direction, and to reduce the time of post-earthquake repairs by confining all the damage to the replaceable brace module.

Traditional gusset plate connections can be designed using the uniform force method (UFM), which has been included in the AISC Manual of Steel Construction since 1992. The UFM was initially proposed by Thornton [5], and is commonly used by engineers in Canada also. The work-point of the brace is commonly located at the intersections of centerlines of the beam and the column (Figure 1(a-b)), and the welds to the beam and the column are designed to satisfy equilibrium such that there is no moment on the column. However, in the proposed alternative connection, the brace module is connected only to the beam, and the work-point is located at the intersection of the column face and the beam centerline. In this case, the beam-column connection is not designed for a moment and a shear tab connection is sufficient for transferring the force (Figure 1c). However, this eccentricity causes an extra moment on the column, and its effect should be considered when designing the columns in braced frames using the proposed alternative connection.

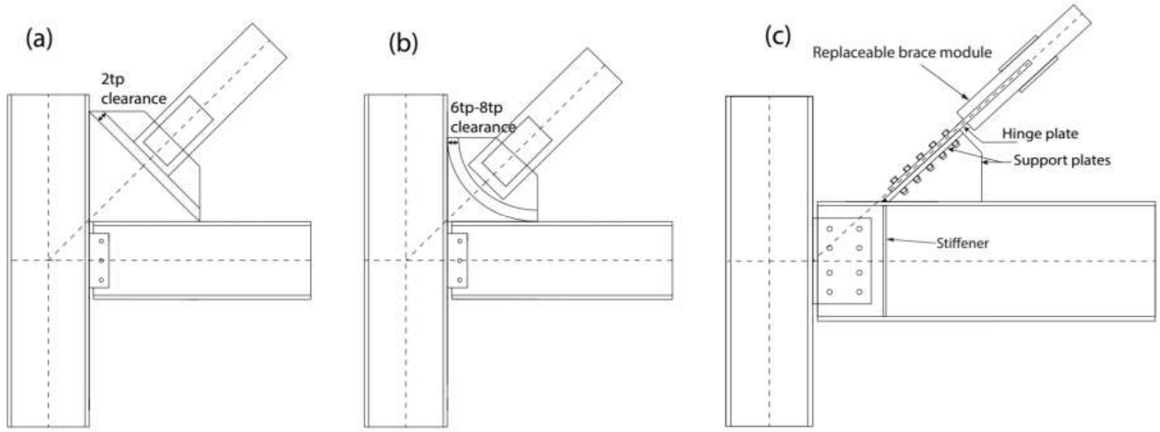

Figure 1. Brace to frame elements connection details: a) Typical gusset plate with linear hinge zone b) Typical gusset plate with elliptical hinge zone c) New proposed connection with replaceable brace module

Design of columns in concentrically braced frames

A special concentrically braced frame (SCBF) is a special class of CBF system that is capable of providing significant inelastic deformation capacity [6]. In Canada, CSA S16-14 refers to moderately ductile concentrically braced frames (MD-CBFs), which have similar design requirements to SCBFs and can be used in regions of high seismicity [7]. An SCBF is designed to transfer the lateral load from the upper stories to the ground primarily via axial forces in braces, beams, and columns. According to AISC 341-16 [8], columns must be designed using the capacity-limited seismic load effect to remain elastic due to brace yielding and buckling. Two analyses must be considered: a) all braces have reached their expected strength in tension or compression, b) all braces in tension have reached their expected strength, and all braces in compression are resisting their expected post-buckling strength. Designing for axial forces from those two analyses might be conservative because such high axial forces may never fully develop, and if they do, it will be only a few times and for very short periods during a seismic event [9]. For that reason, some studies have proposed that the current capacity provisions could be relaxed [9] and others have proposed new combination rules for computing columns' axial demand in braced frames to overcome conservatism in design [10].

In addition to axial force demands, columns are also subjected to moment demands because differences in storey drift ratios of adjacent storeys cause the column to bend. According to AISC 341-16 [6], if the above procedure is used to calculate the axial force on columns, the engineer can neglect the flexural demands on columns, whereas in CSA S16-14 [7] an additional bending moment in the direction of the braced bay of $0.2ZF_y$ must be considered in combination with axial loads. In braced frames with the new proposed connection, the eccentricity that exists between the work-point of the brace and the intersection of the centerlines of the beam and the column (see Figure 1c) is another source of flexural demand on columns. Therefore, there is a need to determine the effects of this eccentricity on moment demands of columns when the new connection is used.

In the present study, a typical six-storey braced frame is designed based on the current design codes [8,11] using two different brace-to-frame connections: (1) conventional connection detailing and (2) the proposed connection detail. In the first phase, as the columns are intended to stay elastic during a seismic event, they are modelled as elastic in order to find the force demands on them. In the second phase, the analyses are repeated but modelling the actual strength of the columns, and the collapse capacities of the frames are evaluated.

DESIGN OF CONSIDERED 6-STOREY SCBF

This study uses a 6-storey SCBF building with a two-storey X configuration, designed for a downtown area of San Francisco, California. The plan configuration of the building is rectangular with braced bays located around the perimeter, as shown in Figure 2. The seismic design loads were calculated based on the maximum spectral intensity (D_{max}) associated with the governing seismic design category in compliance with ASCE/SEI 7-16 [11] using the equivalent lateral force procedure. Table 1 shows the seismic design parameters. The building was designed in accordance with the current AISC seismic design provisions [8]. The assumed effective length of the braces was 70% of the work-point-to-work-point length, and identical column sections were used for every two consecutive storeys.

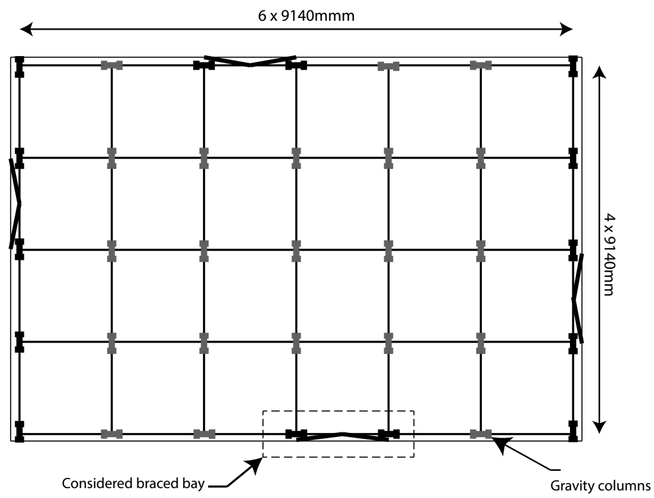

Figure 2. Plan configuration of 6-storey building

Table 1. Seismic design parameters

Parameters	Value
Importace factor	1.0
Site class	D (default)
Short period site coefficient (F_a)	1.2
Short period site coefficient (F_v)	1.7
S_{DS}	1.2g
S_{D1}	0.68g

NONLINEAR MODELLING OF SCBF

The seismic performance of the study frames was evaluated using OpenSees [12]. Figure 3 shows a schematic of the models used in the analyses. Force-based fibre beam-column elements were used to model the inelastic behaviour of the beams and columns, each with 5 integration points to account for the distributed plasticity along the length of each element. Fibre beam-column elements were also used to capture the cyclic inelastic behavior of braces, according to the recommendations of Uriz and Mahin [3]. A uniaxial Giuffre-Menegotto-Pinto steel material with the expected brace yield strength and isotropic and kinematic strain hardening (Steel02) was assigned to each fibre. Fracture corresponding to low cycle fatigue was also considered using the relationships recommended by Karamanci and Lignos [13] to define the input parameters of the steel brace model based on the brace geometric and material properties. Braces were modelled using twenty nonlinear beam-column elements with an initial out-of-straightness in the form of a sinusoidal function with an amplitude of L/1000. When the actual strength of columns was considered, a global out-of-straightness imperfection of L/1000 was introduced to the model using eight nonlinear beam-column elements for each column. The model used in this study could not capture the local web and flange instabilities, but for the stocky columns that were used in this study, these local instabilities are expected to be minimal [14,15].

In the conventional SCBF with typical gusset plate connections, a three-dimensional model was used to allow braces to buckle in the out-of-plane direction. Rigid offsets were modeled at the end of the elements, as shown in Figure 3(a). In columns, these rigid elements extend from the work point to either the physical end of the gusset plate or to the physical end of the beam. In beams, they extend from the work point to the point of 75% of dimension ‘a’ [16]. The gusset plate was modeled using a fibre beam-column element with a length equal to twice the gusset plate thickness and with three integration points. In this model, it was assumed that the gusset plate provides significant strength and stiffness for the beam-column connection, and thus the beam-column connection was modelled as fixed [16].

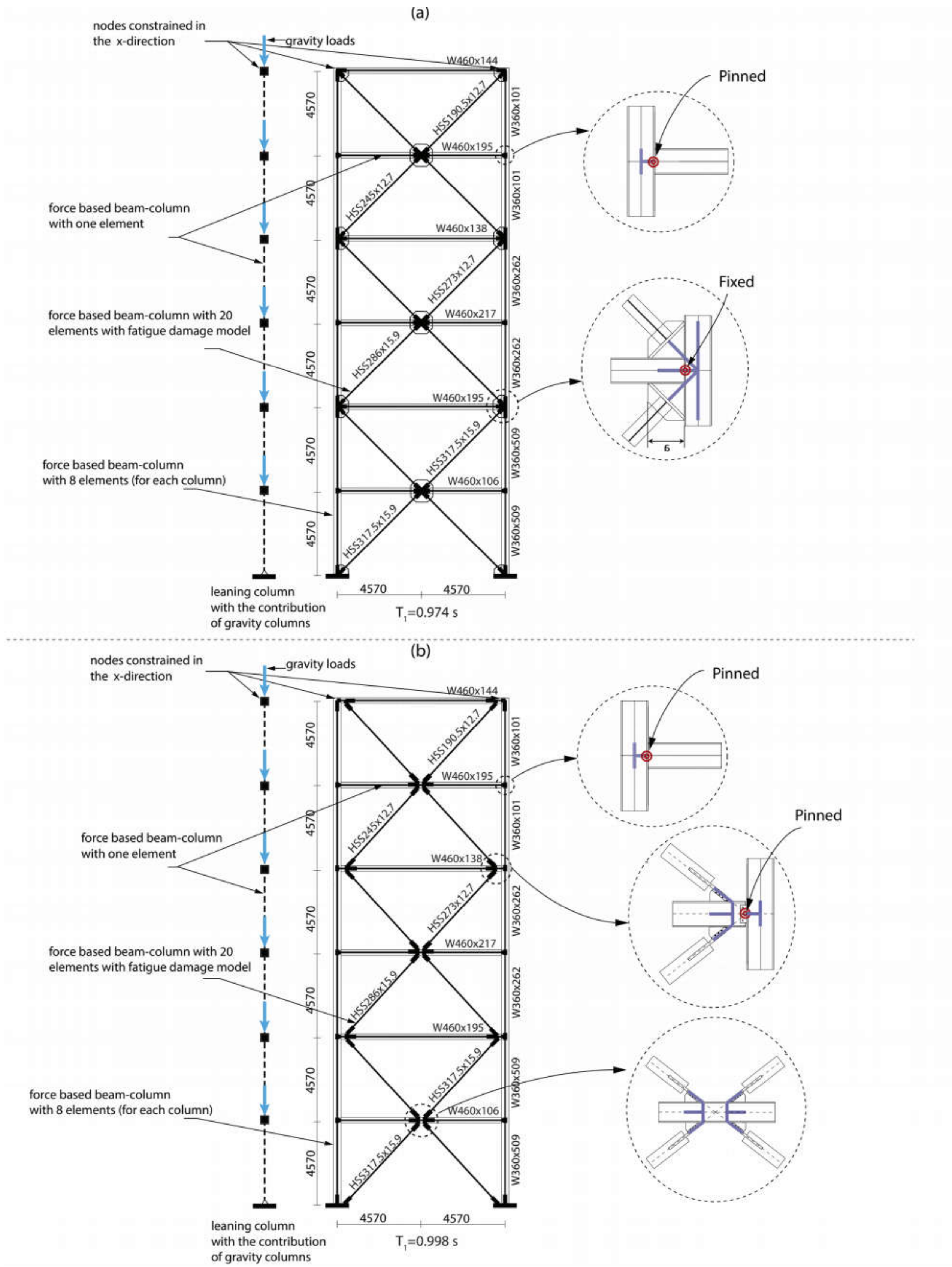

Figure 3. a) Schematic of the numerical model for six-storey SCBF with conventional connection. b) Schematic of the numerical model for six-storey SCBF with replaceable brace modules

Figure 3(b) shows a schematic of the numerical model for a SCBF using the proposed connection detail for replaceable brace modules. In this new connection, the in-plane rotation of the brace is accommodated using a clearance distance equal to twice the hinge plate thickness, so the previous modelling procedure for the gusset plates was also used for modelling the hinge plate. Rigid elements were used to represent the physical size and the stiffening effect of the secondary plates, and connected the intended hinge location to the centerline of the beam. In this model, because the connection is offset from the column face and the traditional gusset plate does not exist, the beam-column connection was assumed to be pinned.

Both models contain a leaning column to represent the gravity framing, as shown in Figure 3. This leaning column, modelled using force-based beam-column elements, was loaded vertically with half of the seismic load of each floor, and the mass tributary to the gravity columns was also lumped at the leaning column nodes. To consider the lateral load resisting contribution of gravity columns, the area, moment of inertia and plastic moment capacity of the leaning column sections were calculated based on all of the gravity columns within the tributary area for seismic weight. The P-Delta geometric transformation formulation was used to simulate P-Delta effects.

Tangent stiffness-proportional Rayleigh damping based on 3% of critical damping in the first and third modes was applied using the committed stiffness matrix.

GROUND MOTION SELECTION AND SCALING

The suite of 44 orthogonal horizontal ground motion components from FEMA P695 [17], which were normalized by their respective peak ground velocities to remove unwarranted variability between records, was used for the analysis. The target spectrum for selecting and scaling the ground motions is the risk-targeted maximum considered earthquake spectrum (MCE_R) of ASCE/SEI 7-16 [11]. The ground motions were scaled to minimize the sum of the differences between the target spectrum and the individual records' spectra within the period range of interest, taken as $0.2T_1$ to $2T_1$, where T_1 is the computed period of the structure. As shown in Figure 4, while the average 5% damped spectrum was in good agreement with the target spectra over a range of periods of interest of greater than T_s=0.58 s, it overshot the target spectrum at smaller periods. After finding the scaling factors for each ground motion, a factor of 2/3 was used to match the average acceleration spectrum to the design basis earthquake (DBE) level.

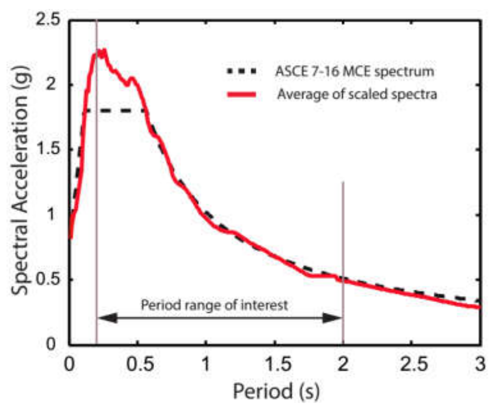

Figure 4. Scaling of the selected ground motions for the 6-story building

COLUMN DEMANDS IN SCBFS

Models with elastic columns

To determine the force demands on the columns, they were modelled to stay elastic during the analyses. Figure 5 shows the median of the maximum P-M ratio during the analyses at DBE and MCE levels at the bottom of each column segment. These values were obtained from the AISC 2016 [18] equations:

$$\begin{cases} When\ \dfrac{P_r}{P_c} \geq 0.2 \rightarrow \dfrac{P_r}{P_c} + \dfrac{8}{9}(\dfrac{M_r}{M_c}) \leq 1.0 \\ When\ \dfrac{P_r}{P_c} < 0.2 \rightarrow \dfrac{P_r}{2P_c} + (\dfrac{M_r}{M_c}) \leq 1.0 \end{cases} \tag{1}$$

where P_r is the axial load demand, P_c is the available axial strength, M_r is the flexural strength demand, and M_c is the available flexural strength. Figure 5 also shows the first term (axial force) and second term (moment) portions in Eq. (1) separately.

At the DBE level, Figure 5(a-b) indicates that the P-M ratios for the columns are slightly larger for the conventional braced frame than for the columns in the SCBF with replaceable brace modules. For both systems, the median P-M ratios are greater at the bottom of the first storey and at the fifth storey, relative to the other storeys. Yielding at the base of the first-storey column happened due to the high rotational restraint (the bottoms of the columns were assumed to be fixed in the models), and P-Delta effects. At the fifth storey, the P-M ratio is greater than one because of the high drift demand at this level due to a greater demand-to-capacity axial force ratio in the braces at this storey relative to the braces in other storeys. While uneven storey drift put large flexural moment demands on the columns, they had not been deigned to stay elastic under combined axial loads and moments. The results also showed that the large storey drifts that developed in the fourth storey also induced high moments at the bottom of the fifth-storey column because the columns were designed as continuous, but with a smaller section assigned to the fifth storey according to the capacity based design procedure.

At the MCE level, Figure 5 (c) shows that all the column segments in the conventional SCBF system experienced P-M ratios larger than one for more than 50% of considered ground motions. This is also the case for the SCBF with replaceable brace modules, as shown in Figure 5 (d), except that the second-storey columns stayed elastic for more than half of the ground motions. Figure 5 shows that at the moment of maximum P-M interaction ratio, the axial force demand is relatively constant at the DBE and MCE intensity levels at approximately 50%-60% of the axial resistance of the column. However, the flexural demand-to-capacity ratios increase significantly at the MCE level relative to DBE level. This increase is more substantial for the SCBF with replaceable brace modules relative to the conventional SCBF, primarily because of the reduced frame contribution to lateral stiffness at higher drift ratios due to the pinned beam-column connection assumption.

Figure 6 shows the median inter-storey drift values at the moment of maximum P-M ratio. The flexural demand-to-capacity ratio becomes larger than 1 when the inter-storey drift ratio is around 1.7% or larger. At the DBE level, maximum P-M ratios often occur at inter-storey drift ratios between 0.5% and 1%. At this range, braces are expected to carry the lateral loads via their axial resistance and there is relatively little flexural demand on columns, while at the MCE level, maximum P-M ratios occur at inter-storey drift ratios of 1% to 2%, when the moment component of the P-M ratio is more significant because of nonuniform drifts in adjacent storeys.

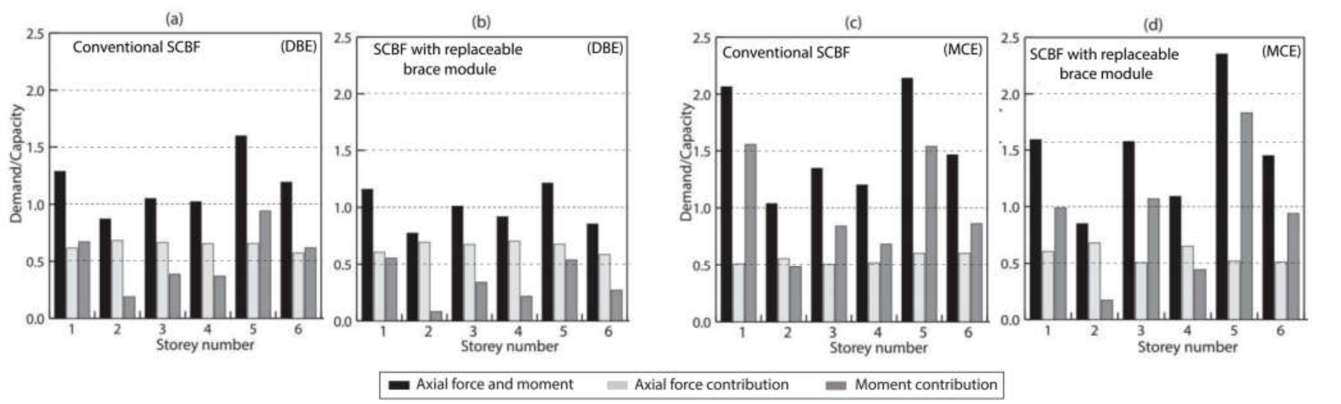

Figure 5. Median demand-to-capacity ratios of columns in SCBF columns: (a)conventional SCBF (DBE), (b)SCBF with replaceable brace modules (DBE) (c)conventional SCBF (MCE), (d)SCBF with replaceable brace modules (MCE)

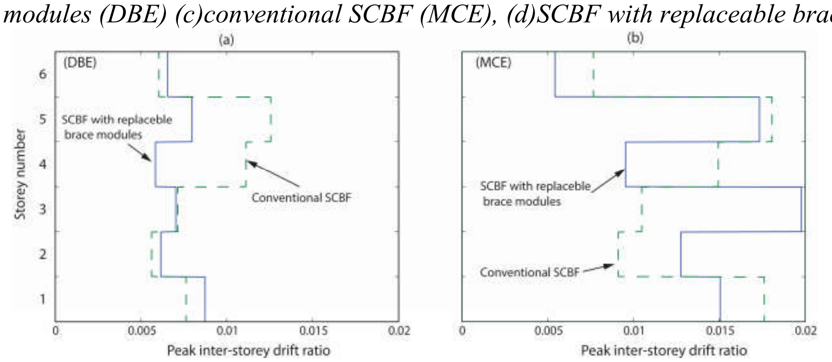

Figure 6. Median inter-storey drift ratio: (a)DBE and (b)MCE

Models with inelastic columns

In this phase of the study, force-based nonlinear elements were used to model column yielding and buckling. Some of the ground motions caused large inter-storey drifts that would be large enough to result in flexural deteriorations due to local instabilities in columns, which the model is not able to capture. Columns that satisfy the local slenderness limits for highly ductile members as per AISC 341-16 [8], on average, will lose 20% of their maximum flexural strength and experience 50 mm axial shortening at about 6% inter-storey drift ratio when subjected to collapse-consistent lateral loading [15]. For that reason, collapse was defined as a state at which the maximum storey drift reaches 6%.

To evaluate the collapse fragility with these more sophisticated models, a multiple stripe analysis (MSA) [19] was conducted using three stripes at the DBE, MCE and 1.5MCE levels. Figure 7 shows the resulting fragility curves for the considered frames. The FEMA P695 [17] methodology was used to evaluate the collapse capacity of the numerical models. In this methodology, the collapse potential of a performance group, which consists of at least three building archetypes, is acceptable when their average probability of collapse is less than 10%, with no one exceeding 20%. To evaluate this criterion, the adjusted collapse margin ratio (ACMR) was computed using Eq. (2), and was then compared to the acceptable collapse margin ratio corresponding to a 10% collapse probability limit ($ACMR_{10\%}$) for a given total system uncertainty.

$$ACMR = \frac{\hat{S}_{CT}}{S_{MT}} SSF \qquad (2)$$

In Eq. (2), $\hat{S}_{CT}$ is the median collapse intensity, S_{MT} is the MCE ground motion spectral demand, and SSF is the spectral shape factor to account for the frequency content of the ground motion record set. Table 2 summarizes the results of the evaluation process. For the SCBF with replaceable brace modules, $\hat{S}_{CT}$ is smaller than for the conventional SCBF. However, it is also compared with a smaller acceptable $ACMR_{10\%}$ because it has less record-to-record uncertainty (β_{RTR}). Neither system passes the FEMA P695 collapse criterion. This implies that designing the columns of this example SCBF system according to current provisions in AISC 341-16 [6], which only consider axial force demands, does not produce an acceptable collapse capacity regardless of the connection type.

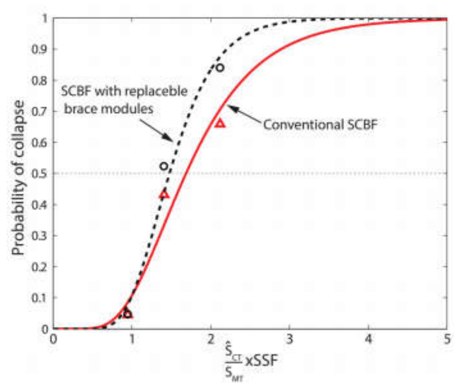

Figure 7. Collapse fragility curves

Table 2. Collapse results

Considered model	β_{DR}, β_{TD}, β_{MDL}	β_{RTR}	SSF × S_{CT}	S_{MT}	β	ACMR	Acceptable $ACMR_{10\%}$	Pass/Fail ratio
Conventional SCBF	0.2	0.43	1.67g	1.03g	0.55	1.62	2.00	**0.80**
SCBF with replaceable brace modules	0.2	0.31	1.47g	1.03g	0.46	1.43	1.80	**0.80**

SUMMARY AND CONCLUSIONS

This paper investigated the influence of an innovative new connection detail on the column force demands in a six-storey SCBF. First, the columns were modelled to stay elastic during the analyses to find the axial and flexural demands on them.

Then, the columns were modelled with nonlinear elements to capture their actual behaviour. The results revealed that there is only a slight difference between the maximum P-M interactions of the columns of a conventional SCBF and an SCBF with replaceable brace modules, and this difference is attributable to the reduced frame contribution in the latter case due to assumed pinned beam-column connections. The results also indicate that analysis approaches that only take into account the axial demand for designing the columns are not sufficient and should be reevaluated. Future work will investigate how to design the columns in a braced frame in a way that will ensure that the system will pass the collapse criterion and that the columns will avoid significant yielding in seismic events.

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