

Seismic Design of Eccentrically Braced Space Frame

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ABSTRACT: Case studies of the design of a building employing three dimensional eccentrically braced frames as lateral force resisting system are discussed. The floor framing system consists of composite beams supported by long span trusses, which are then supported by exposed composite columns. The lateral system is a dual system. The primary lateral system is made of four EBFs supported by the eight composite columns and folded around the four corners of the main building. The secondary lateral system is a moment resistant space frame consisting of the floor trusses and the composite columns. Nonlinear pushover and earthquake response history analyses indicate that the dual structural system possesses substantial reserve strength over the code prescribed minimum lateral loads. The member yielding sequence and the distribution, as well as the extent of the plastic hinge rotations observed from the analyses illustrate that the building is likely perform satisfactorily under severe seismic events.

Keywords: Seismic Design, EBSF, Modelling, Pushover Analyses

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INTRODUCTION

During the last decade, eccentrically braced frames (EBFs) have involved (Engelhardt et al., 1989) and become fully codified (ICBO, 1994) as an effective earthquake resistant framing system for building structures. In particular, a dual structural system employing EBFs around the building perimeter has gained wide acceptance as an efficient choice, as it economically satisfies both architectural and the structural requirements. In most case, a complete EBF, including the link beams, braces, and columns, is located in the same vertical plan. While the EBFs are often the primary lateral system for a building, they are usually built within partition walls and thus are seldom part of the architectural statement (ICBO, 1994).

In this paper, the structural design of an office building employing exposed three-dimensional (3D) EBFs as the primary lateral force resisting system discussed in detail. The building has 13 stories above grade and four levels below grade. The typical above grade floor-to floor height is 4.2 m. A novel design of an eccentrically braced exoskeleton is adopted as the primary lateral system for the building.

The above-grade structure consists of a concrete platform between the ground level and level 3, and a glass box above level 3 with exposed steel framing supported by eight steel and reinforced-concrete composite columns. The typical floor framing system consists of composite beams supported by long span trusses, resulting in a column free interior space. The primary lateral system is made of four 90° folded EBFs supported by the composite columns at the four corners of the main building. An isometric view of the superstructure is shown in Figure 1. The secondary lateral system is made of MRFs in both principal directions. The MRFs are formed by connecting both of the top and bottom chords of the floor trusses to the composite columns.

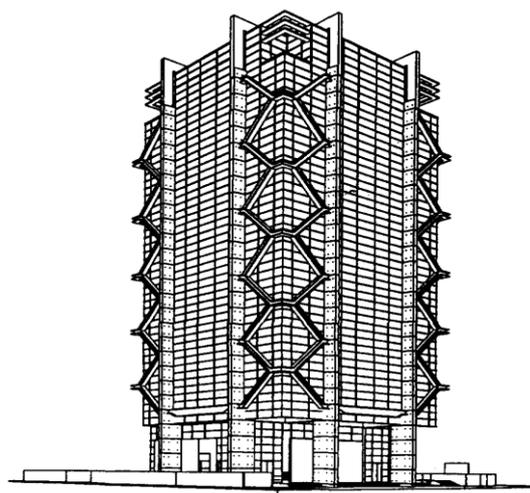


Figure 1. Isometric view of building

The design of the building structure is governed by the combination of gravity and earthquake lateral load effects. In addition to the conventional 3D elastic analyses, the post yielding behavior of the lateral system assessed using nonlinear pushover and earthquake response history analyses. The nonlinear analyses are conducted to examine the structural performance in ultimate earthquake events, and to verify the adequacy of the structural design following the code prescribed procedures (Loh et al., 1995).

Seismic bracing has long been a staple of seismic-resistant design since it provides reasonable architectural and mechanical flexibility compared to wall systems and can provide satisfactory performance with a small number of braced frames. Conventional concentrically braced frames (CBFs) (Figure 2a) readily provide high levels of monotonic stiffness and strength. However, they often exhibit strength and stiffness degradation during their

plastic cyclic response (Popov et al., 1976; Tremblay, 2001). As observed in past earthquake damage (Bertero et al., 1994; AIJ, 1997; Nakashima, 2000; Bruneau et al., 2011) and in laboratory tests (Khatib et al., 1988; Roeder et al., 2011), CBFs are significantly affected by the poor performance of the braces in compression, occurrence of fracture in the brace after a small number of plastic load reversals, and complicated detailing of the gusset plate connection at the ends of the brace member. Additionally, CBF braces can typically only be arranged in a few configurations and provide few design variables to achieve desirable stiffness, strength, and ductility and design is controlled by compressive brace buckling behavior, all of which inhibit the potential seismic performance of the lateral system. Properly detailed eccentrically braced frames (EBFs) (Figure 2b) that utilize ductile shear or flexural links placed between eccentric brace connections can provide an attractive combination of strength and ductility (Roeder and Popov 1978; Popov and Engelhardt, 1988). The ductile links, however, are relatively complex elements that experience a combination of shear and flexural effects and therefore require careful design treatment (Okazaki et al., 2005). Additionally, the EBF links are currently placed within the gravity load resisting systems (floor beams) and can cause significant damage to the beam and surrounding floor slab that can be costly to repair.

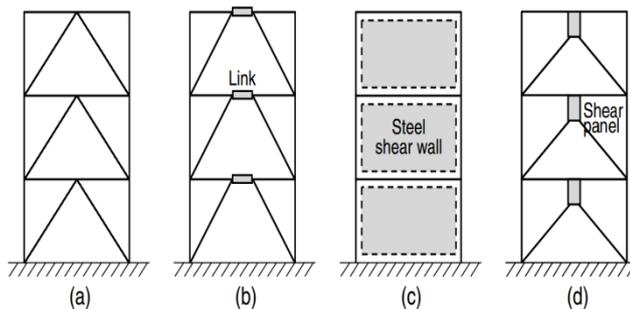


Figure 2. Seismic load resisting steel systems: (a) concentrically braced frame (CBF); (b) eccentrically braced frame (EBF); (c) steel panel shear wall (SPSW) system; (d) shear panel system

Structural system

The structural framing from the foundation to level 3 is of reinforced concrete. A massive reinforced-concrete floor system at level 3 is incorporated as the transition between the reinforced-concrete structure below level 3 and the steel framing system above level 3. The reinforced concrete waffle slab/girder floor framing plan for level 3 is shown in Figure 3a.

The floor framing (level 4-13) consists of wide flange steel beams with metal deck and regular weight concrete fills. The floor framing system for typical office floors (level 4- 13) is shown in Figure 3b. The wide flange steel beams are supported by four steel trusses. The trusses span 26.6 m in each direction and are supported by eight exposed composite columns outside the building façade, leaving a column free office space for typical floors.

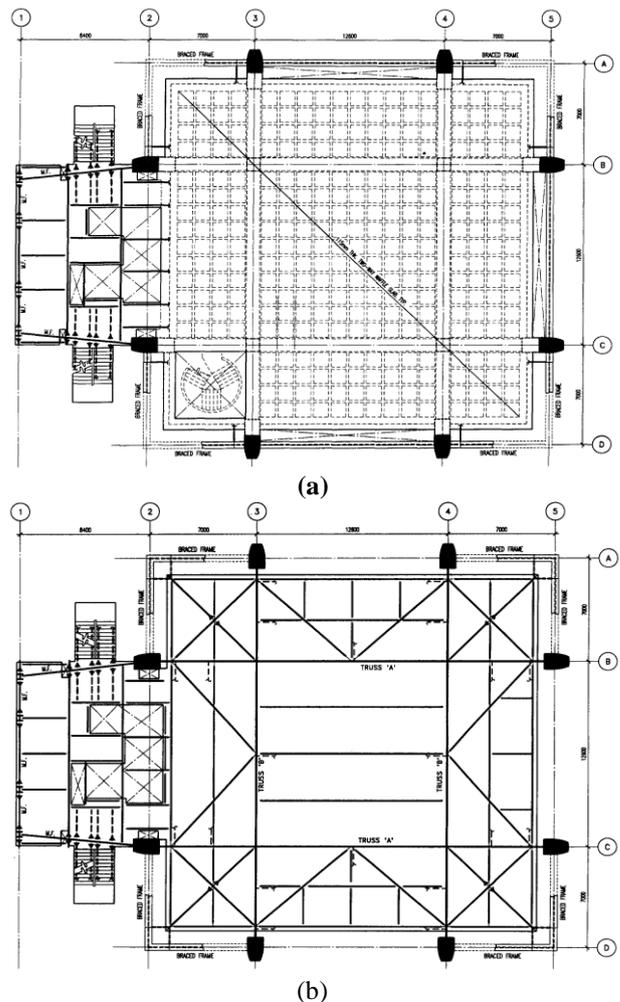


Figure 3: (a) Level 3 framing plan, (b) Typical floor (levels 4-13) framing

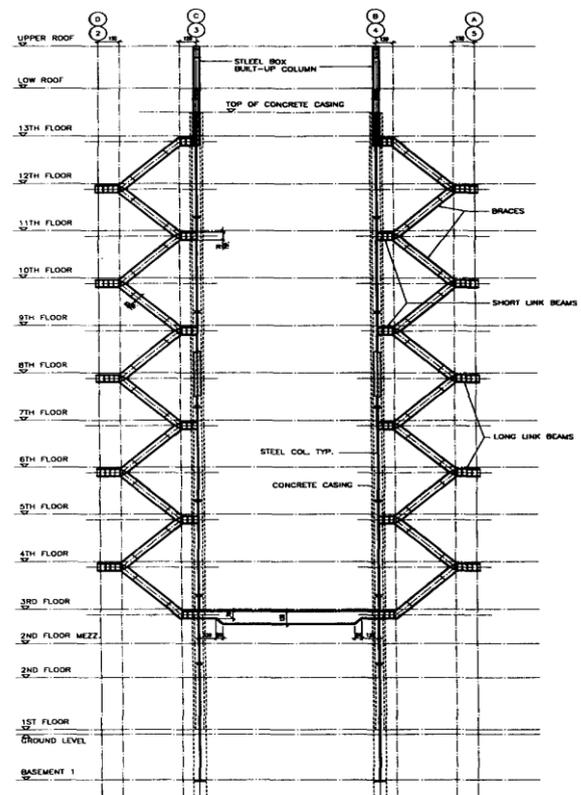


Figure 4. Eccentrically braced frame elevation

From the ground level to level 3, the lateral system consists of reinforced- concrete space frames. The space frames are made by the waffle slab girders, interior reinforced- concrete columns and the perimeter composite columns. The primary lateral load resisting system above level 3 consist of four 90° folded steel eccentrically braced space frames, one at each corner of the main office floor. A secondary system, consisting of the four floor trusses as noted above “moment connected” to the composite columns, is also provided for redundancy. The “moment connection” of the floor trusses to the composite columns is achieved by connecting both top and bottom chord of the trusses to the column. The four corner eccentrically braced space frames are architecturally exposed and are located approximately 900 mm away from the building façade, hence offsetting from the main floor slab. The corner eccentrically braced space frame do not support any gravity load except own weight.

An elevation of the eccentrically braced space frames at the building perimeter is shown in Figure 4. Each EBF is folded into a 90° angle at the corner of the link beams to fit into the four corners of the building. Therefore, a series of link beams are created at the building corners with one half of the link beam length shown at each face of the building. This link beams are referred as corner link beams for the content of this paper. The corner link beams are expected to experience shear yielding in severe earthquake events and to serve as the primary energy dissipation elements for the building. There is also a second set of link beams, designated as the architectural link beams, which connect the braces to the composite columns. The existence of the second set of link beams is mainly for the architectural expression on the elevations. To avoid complication of the structural behaviour, the architectural link beams are designed to remain elastic under the ultimate loading conditions.

A continuous diaphragm wall around the basement perimeter is provided for retaining lateral soil pressure and transferring the wind and seismic lateral loads to the foundation. Due to the soft soil condition and the large concentrated loads imposed by the superstructure, a pile foundation system is adopted.

Seismic loading

The project site is located in the Taipei Basin, where there are deep and soft soil deposits with a long predominant period. According to the Building Technology Standard (BTS) (MIRC, 1995), which is the local building code in Taiwan, the site is in a moderate seismicity region. A representative recorded grounded ground acceleration history in the project site (Figure 5) shows only moderate peak ground acceleration (PGA). However, the duration of strong ground shaking is fairly long and the ground acceleration are rich with low frequency contents in the range of 0.5- 1.0 Hz. as an example of recorded ground acceleration at stiff soil site, Figure 6 shows the elastic response spectrum derived using the 1940 El Centro Earthquake N-S component (PGA=0.34g) and 5% damping ratio. For comparison, the elastic response spectrum derived (assuming a 5% damping ratio) using the aforementioned ground acceleration records (scaled to the same PGA of 0.34g) is also plotted in Figure 6, showing much more severe structural responses.. Regardless of the PGA values, it is

expected that the earthquake response of long period (fundamental period exceeds 1.0 s) structures is comparable to that in seismic zone 4.

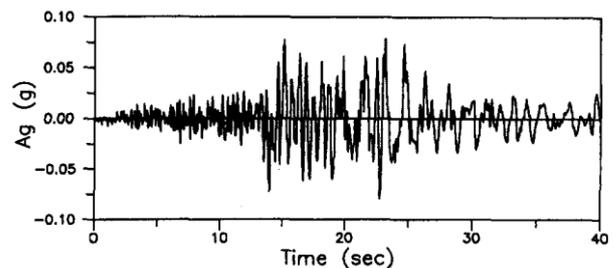


Figure 5. Recorded ground acceleration in Site

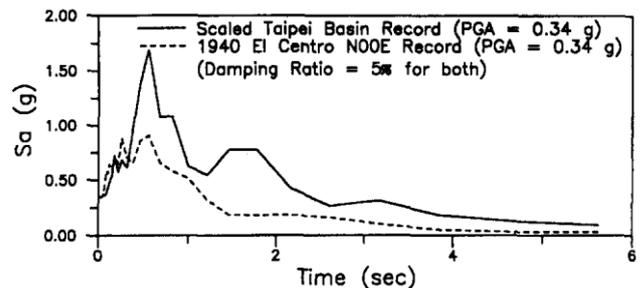


Figure 6. Comparison of elastic response spectra

Elastic analysis and design considerations

The analysis of the building structure was started with a conventional elastic stress analysis. A 3D structural model, which includes both above-grade and basement structures, was developed using the design material properties and member sizes. A general purpose finite-element static and dynamic structural analysis program, SAP2000, was used to perform the analysis (Wilson et al., 1992).

Structural modelling

Among all structural members in the building, the following elements in the building have been carefully modelled in the structural model to reflect actual conditions of construction.

Diaphragm to frame connection

Due to the offset of the eccentrically braced frames to the main floor slab, the EBFs are not attached to the floor slabs directly. The composite columns are connected to the floor slabs only through the top chord of the floor trusses (Figure 7). To compute the forces in such connections, a master node is defined at the center of mass at each floor, and a series of beam elements (referred to as diaphragm connectors) at the end of each floor truss are defined.

These diaphragm connectors have one end rigidly connected to the diaphragm (slaved to the master node) and the other end connected to the composite column, which is not rigidly connected to the diaphragm (not slaved to the master node). The model system described here is to ensure an appropriate modelling of the forces transferred from the floor diaphragm to the braced frames. Thus, the internal forces in the diaphragm connectors are correctly calculated for various loading conditions.

Eccentrically braced space frames

All nodes associated with the eccentrically braced frame including link beams, braces, and composite columns are not slaved to the master node and have independent degrees of freedom. The member deformations and internal forces, including axial load, biaxial shear and bending, as well as axial twisting, are calculated to thoroughly understand the actual behaviour of those elements. Because of the relatively low torsion rigidity of wide flange sections (which are used for all link beams) and the minimal warping restraint condition at the corner point, the torsion moment developed in corner link beams is nearly zero. As a result, the bending moment developed at the corner end of the link beams is negligible.

Floor truss to composite column connections

The floor truss to composite column connections are being built in two steps: the top chord of the truss is first connected to the steel erection column, and the bottom one is connected to the concrete encased composite column after the construction of the concrete slabs. There are two advantages of such a construction sequence. One is that the design of majority of truss members is controlled by gravity loads, which do not induce tensile and compressive stress reversals, so that the design of truss members can be easily optimized. The other is that the flexural strength demand imposed by the gravity load is limited (no fixed end moment at the truss ends due to the dead load), and therefore, a strong column weak girder (weak truss in this case) design has been achieved without much difficulty. Two-stage elastic analyses, one for structural self-weight only without truss bottom chord connected, have been performed to consider these conditions.

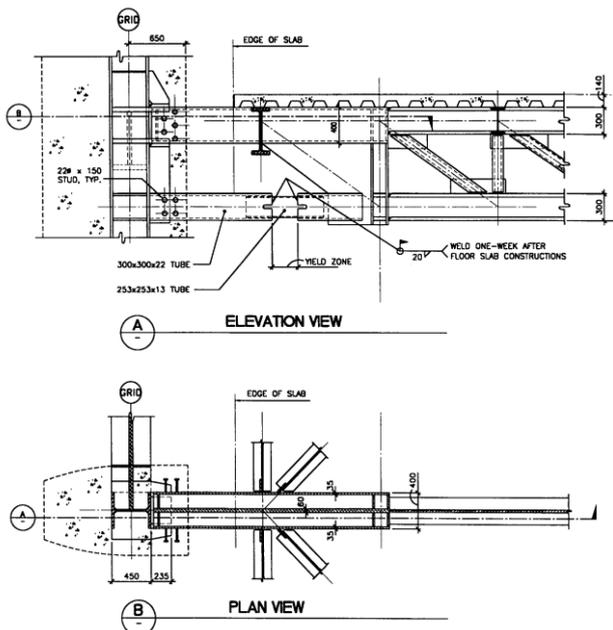


Figure 7. Details of floor truss to composite column connection: (A) elevation view; (B) plan view

Observed elastic behaviour

The elastic analysis shows that the fundamental periods of the building are 1.67 s in the X-direction (parallel to grid line A), 1.64 s in the Y-direction (parallel

to grid line 1), and 1.03 s in torsion. It is interesting to note that the fundamental modes are in the principal directions of the building. Ignoring the existence of the service area and the moment frame in that area for the purpose of discussion such behaviour can be explained as follows. Each of the four corner EBFs can be viewed as a cantilever frame above level 3.

The two columns are coupled by a series of 90° folded link beams and eccentric braces. When subjected to lateral loads, each EBF behaves as a slender cantilever. Thus, ignoring the bending stiffness of individual columns the lateral stiffness of an EBF can be estimated as

$$I = A \times \frac{d^2}{2} \quad (1)$$

Where A = cross-sectional area of one composite column; and d = distance between the two composite columns in the direction that the EBF is loaded. If the lateral stiffness of such an EBF in the X- or Y- direction is referred as I_0 , then the stiffness is $2I_0$ when it is loaded in the plane connecting the two end columns. The stiffness of the same EBF is zero when loaded in the direction perpendicular to the plane of the two columns. Thus, the total lateral stiffness of the four EBFs is the same ($4I_0$) whether the building is loaded in a principal direction or in a diagonal direction

Design load conditions for corner link beam

The corner link beams are designed for the most critical combination of the gravity and earthquake internal loads. Particularly, the shear capacity of the corner link beams is determined based on the following loads combinations:

$$V = V_{DL} + V_{LL} \pm V_{1.0EQ_x} \pm V_{0.4EQ_y} \quad (2)$$

or

$$V = V_{DL} + V_{LL} \pm V_{0.4EQ_x} \pm V_{1.0EQ_y} \quad (3)$$

In which DL , LL , EQ_x , and EQ_y are the dead load, live load, and earthquake loads in X- and Y- direction, respectively. For a given direction where the earthquake loads are applied, the link beam shear force used in the aforementioned load combinations is the larger of that calculated in the static earthquake load case and in the scaled (to the same base shear as that in the static earthquake load case) spectral analysis case. It is live loads are minimal. Thus, the most critical earthquake load combination, resulting from 100% load in one direction and 40% load in the orthogonal direction, governs the design of these link beams.

Lateral system design and detailing

The lateral force resisting system is designed as a dual system, considering the corner EBFs as the primary system, and the frames made of floor trusses and composite columns as the secondary system. The design requirements prescribed in the UBC are followed.

The corner link beams are the primary energy dissipation elements for the building, and their required shear capacities are first determined based on the elastic static and dynamic analyses as discussed previously. Aimed at achieving a shear yielding behaviour, the flexural capacities of the corner link beams are then

determined. Typical details for the corner link beams and their connections are shown in Figure 8. Each link-to-brace joint is laterally restrained at both top and bottom flanges of the link beam by two square tubes. In the construction of the 3D EBFs, the link beam corner connections and the link-to-brace connections are all shop welded. Field splices are made for braces only, where no yielding is ever expected.

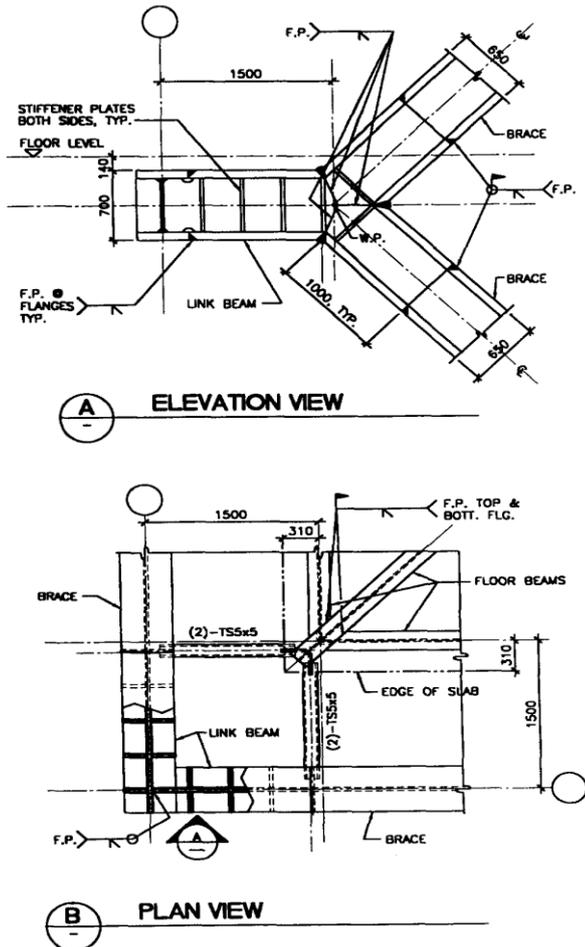


Figure 8. Typical corner link beam detail: (A) elevation view; (B) plan view

Inelastic analysis and post yielding performance

Inelastic analyses have been performed to evaluate the post yielding behaviour of the lateral load resisting system. The items investigated included the possible yielding sequence and the maximum plastic rotation of the corner link beams, the distribution of the lateral load between the primary and the secondary systems, and the global yielding mechanism, if developed, in the whole lateral load resisting system. Two types of inelastic analysis were performed, pseudo static pushover analyses and dynamic response history analyses. Both types of analyses are conducted on a two-dimensional (2D) model using a general purpose static and dynamic inelastic analysis program, DRAIN2DX (Parkash et al., 1993). The close match of the first two mode shapes and periods (the periods are slightly shorter than that from the 3D model as the 2D frame is fixed at the ground level) (Figure 9) between the 2D model and the 3D SAP2000 model confirms the accuracy of the 2D approximations (Powell, 1993).

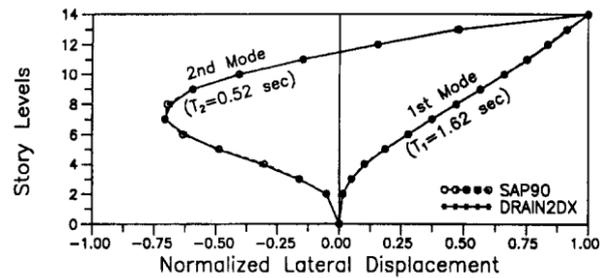


Figure 9. Mode shape comparison of 2D models

Determination of Element Yielding Capacity

A bilinear (with a 2% strain hardening rate) moment-rotation relation is adopted to model the inelastic behaviour for all of the frame members. The member yielding strength is defined as the plastic capacity for steel members and as the nominal strength for reinforced-concrete and composite members. Particularly, the yielding capacity of floor trusses and link beam is further discussed.

Truss end flexural capacity

The floor trusses are made of many individual members, and it is not practical to model every element in each truss for the purpose of inelastic analysis. Therefore, each truss is represented by an equivalent wide flange steel beam having equivalent stiffness and plastic capacity as those of the truss in the analytical model. The plastic capacity at the end of a truss consists of two parts, and can be expressed as

$$M_{p,truss} = F_{y,bc} \times d + M_{p,tc} \quad (4)$$

in which d = center line distance between the top and bottom chords; $F_{y,bc}$ = axial yielding force in the bottom chord; and $M_{p,tc}$ = plastic moment capacity of the top chord. At the end of the trusses, the top chord has a substantially larger section than the bottom chord (it is required to carry the vertical shear force and to transfer the horizontal diaphragm force to the frame), and the bottom chord is detailed such that a tension/compression yielding can be developed (Figure 7). It is also noticed (Figure 3) that the bottom chord of the trusses has adequate lateral bracing. Therefore, it is justified that the truss plastic capacity so defined will closely represent the actual strength.

Link Beam Shear Capacity

There are two groups of link beams, the corner link beams and the architectural link beams. The corner link beams are expected to develop shear yielding when the shear demand exceeds the plastic capacity. However the current version of the DRAIN2DX program does not have an element modelling the shear yielding behaviour. Therefore, an equivalent plastic moment capacity is defined for each link beam to model the shear capacity indirectly. For the corner link beams, since the moment at the corner point is always zero, an equivalent plastic moment capacity M_{ep} can be defined as

$$M_{ep} = V_s \times L \quad (5)$$

where V_s = plastic shear capacity; and L = length of the link beam and braces to the corner point. The actual flexural capacity of these link beams is substantially higher than the aforementioned equivalent capacity, and is

not explicitly modelled. The architectural link beams have nonzero moment at both ends of the beams, and the moment developed at the two ends are, in general, different. To define the plastic shear capacity, the condition of formula 6 needs to be satisfied,

$$V_s = \frac{M_{ep,B} + M_{ep,C}}{L} \quad (6)$$

where $M_{ep,B}$ and $M_{ep,C}$ = equivalent plastic flexural capacities at the end connecting to the braces and to the column, respectively. The actual flexural capacity is again higher than those equivalent values. For simplicity a fixed ratio of $M_{ep,B} / M_{ep,C}$, which is determined as the average ratio for all architectural link beams from the elastic analysis, is used to define the equivalent plastic flexural capacities, a shear yielding in the link beam is then implied

Pseudo Static Pushover Analysis

In this analysis, the ultimate state is defined as the roof level lateral displacement having reached the value equal to 1.5% of the building height. This roof drift ratio is considered a suitable benchmark for controlling the lateral displacement of a building. The result of the nonlinear response history analyses demonstrate that such a roof drifts is appropriate for the subject building.

Figure 10 shows the total base shear versus the roof lateral displacement relationships. It is observed that the structural system has adequate reserved strength compared with the code minimum requirement. The member yielding sequence obtained from the pushover analysis indicates that the corner link beams yield (in shear) first. Hinging is then developed at the truss ends in the secondary frame and at the bottom of columns as the lateral loads increase. This yielding sequence has demonstrated a desirable lateral load resisting mechanism, which is intended for the subject dual system.

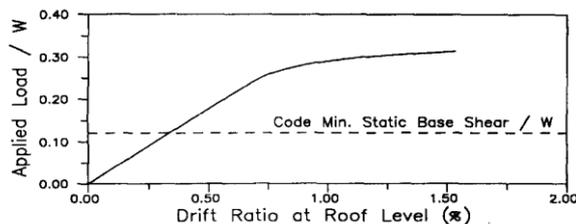


Figure 10. Pseudo static pushover analysis

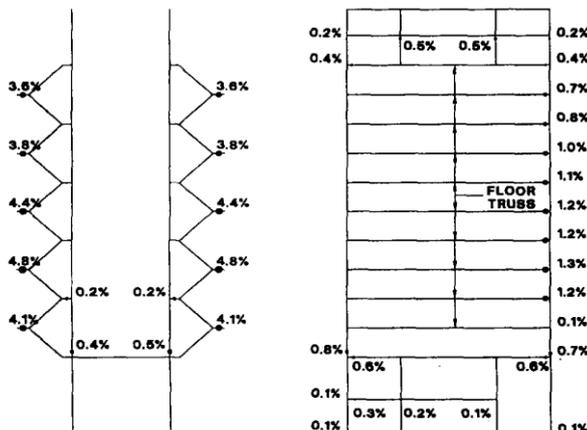


Figure 11. Pseudo static pushover analysis plastic hinge location and rotation: braced frame on grid <D>; secondary frame on grid <C>

The yielding pattern and the maximum plastic hinge rotation at the ultimate state are shown in Figure 11. It is evident that no brace or architectural link beam yielding has occurred. There is also no column yielding except at the ground level and above the level 3 platform. The calculated maximum link beam rotation is 4.8% radian, less than that allowed (6% radian) in the UBC. Thus, the static pushover analysis has confirmed the fundamental assumptions made regarding the performance of the structural system.

Nonlinear response time- history analysis

Using one of the ground acceleration records (Figure 5) obtained in the vicinity of the project site, a dynamic time- history response analysis is performed. The roof level lateral displacement history and the seventh story (where the maximum Inter-story drift occurs) Inter-story drift ratio history are shown in Figure 12 and 13, respectively. The maximum roof lateral displacement is about 500mm, which is less than 0.9% of the building height. During the entire response history, the maximum Inter-story drift ratio is less than 1.2%, implying only limited structural damage is expected in the most severe earthquake event. Figure 14 shows the base shear history obtained from the time- history analysis. Finally, the plastic hinge distribution and the maximum plastic rotation for each hinge are shown in Figure 15. The maximum link beam rotation is about 1.1% rad, and the plastic hinge rotations in all other members are insignificant. It is demonstrated by this analysis that the building is expected to have satisfactory performance in severe earthquake events.

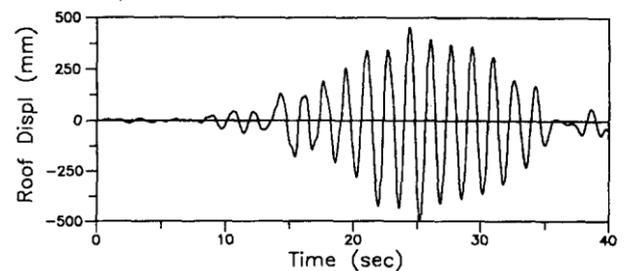


Figure 12. Response history analysis

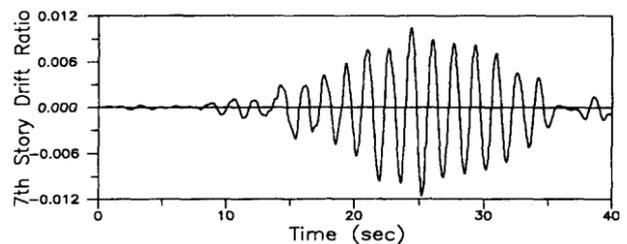


Figure 13. Response history analysis (roof level lateral displacement, seventh story drift rotation)

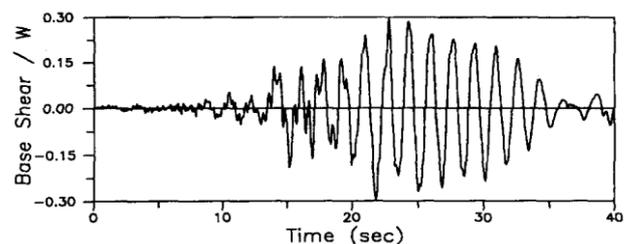


Figure 14. Response history analysis

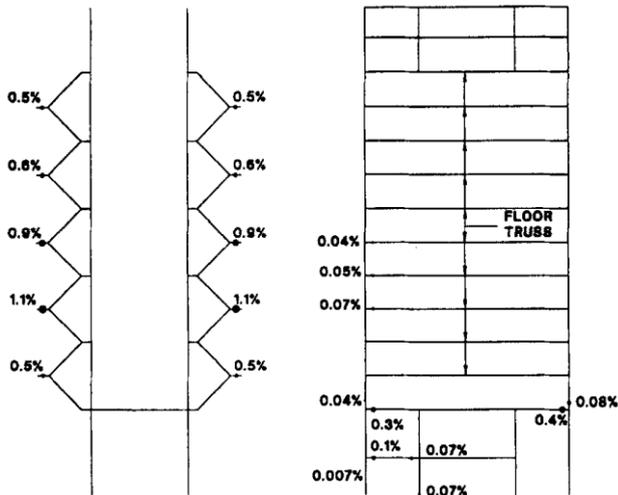


Figure 15. Response history analysis plastic hinge (total base shear) location and rotation: braced frame on grid <D> and secondary frame on grid <C>

CONCLUSION

A novel design of the dual structural system consisting of exposed eccentrically braced space frames and moment resisting space frames composed of floor trusses and composite columns is presented. The structural design is performed based on the state of the art knowledge of earthquake resistant design. Code capacity design requirements for EBFs and the strong-column weak girder requirements for MRFs have been strictly followed. Post yielding behaviour of the structure has been investigated using a 2D nonlinear structural model. The techniques adopted in approximating the 3D EBFs by a 2D model are discussed. Analytical results indicate that the 2D model adequately predicts the dynamic characteristics of the 3D structural system.

Result of nonlinear pushover analysis illustrate that the Post yielding strength of the system is significantly higher than the code prescribed minimum. Shear yielding in the corner link beams is developed before plastic hinges forming at the ends of trusses. Results of nonlinear dynamic analysis further indicate that the nonlinear deformation demand imposed on the structural members under ultimate earthquake events is limited. Thus, it is anticipated that the proposed structural system will perform satisfactory in resisting severe earthquakes.

The case studied presented indicate that the code prescribed design procedures for EBFs, which utilizes the capacity design principles and aims at achieving a controlled post yielding structural behaviour, is suitable for producing proficient design for EBF systems. In addition, a post yielding evaluation of a structure using representative analytical models provides valuable in-depth information of the ultimate behaviour of a structure. Such analyses help to verify the earthquake resistant performance of a structure and to detect any deficiency it may possess, and therefore, is highly recommended for the earthquake resistant design of any unconventional structural systems.

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