# PERFORMANCE BASED ANALYSIS AND MODELING OF A DUAL SEISMIC FORCE-RESISTING SYSTEM

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## SUMMARY

The performance-based analysis and modeling of a 13-story office building with a dual seismic force-resisting system, eccentrically braced frames (EBFs) and moment resisting frames (MRFs), is presented. The seismic system was developed for the CEC corporate headquarters building in Taipei, Taiwan. The building was designed to resist the earthquake forces specified by the 1995 Taiwan building codes and satisfied the detailing requirements prescribed in the 1994 UBC. The case studies were performed using nonlinear static pushover procedures to develop the system capacity curve. It is found that inelastic behaviour of the building is primarily due to shear yielding of the EBFs links and yielding of moment-resisting elements in the trusses. For design verification, a non-linear time history analysis with recorded ground accelerations in Taipei basin was used to check the inter-story drift distribution and ductility demands at the hinge points of the system yield mechanism. The results from the non-linear static pushover analysis show that the EBFs in the dual system was able to develop inter-story drift ratios of the order of 1.5% to 2.0%, nearly uniformly over its height, while limiting the plastic shear strains in the EBFs links to less than 5.0% radians.

## INTRODUCTION

Eccentrically braced frames (EBFs) are widely used style of steel buildings because of its viability for earthquake resistance [2]. A properly designed EBFs core coupled with moment resisting frames (MRFs) would offer the best choice of lateral stiffness and steel framing weight incorporated with the structural design objective and architect-defined constraints. The design of seismic-resistant EBFs is based on the concept that, during an extreme earthquake event, yielding and damage to the structure must be limited primarily to the link beams, which will deform inelastically with significant ductility and energy dissipation. For most EBF configurations, the link beams, brace members and columns are often located within the same vertical plane. Past EBF research has concentrated on short links, with limited information available on the real behaviour of those other than short links.

This paper summarizes performance-based seismic analysis and modeling of a 13-story office building consisting of exposed three-dimensional EBFs as the primary seismic force resisting system. The core, at the four corners of the main building, was braced with eccentric braced frames in the direction of each principal axis by folding the EBF link beams with an angle of 90 degree, and then coupled to the eight exterior composite columns with moment-resisting frames to improve its efficiency in controlling seismic drift. Four long span steel trusses with a boxed configuration were designed to create column-free office space for each typical floor. Additional two-way reinforced concrete space frames were placed at the ground level to adjust the centre of rigidity and control drift due to torsional movement. The subject building has been constructed in Taipei, Taiwan, with its foundation on deep soil deposits in the area of high seismic hazard. The proposed EBF configurations and the project site necessitated a comprehensive dynamic analysis of the building. Both nonlinear static pushover and nonlinear response time-history analysis with recorded ground acceleration in Taipei Basin were performed to establish the inelastic rotation demand on the folded links. The analysis was

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intended to identify the potential failure modes of the folded links, and the sequence of member yielding in order to assure that the link behaviour is controlled by the more desired shear yielding mechanism. Recommendations for the seismic design of such folded links are also provided.

## STRUCTURAL SYSTEM DESCRIPTION

The architectural design of the building resulted in an approximately symmetrical distribution of floor mass and structural framing [8]. However, the tower columns are typically spaced at 26.6m centre-to-centre in the principal framing direction and 12.6m in the non-framing direction to create column-free interior office space (and thus maximum future flexibility). These tower columns are composite above ground level, and reinforced concrete from foundation to ground floor to mesh with the basement reinforced concrete construction. Reinforced concrete framing was used for the floor plates from foundation level up to the third floor. The office floor framing system from third floor to roof is a composite metal deck and regular weight concrete fill with wide flange steel beams, typically spanning around 3m centre-to-centre, that act compositely with the floor slab system. The steel floor beams are supported by long-span trusses that run in two orthogonal framing directions, as shown in Figure 1. A reinforced concrete waffle slab/gird framing system at the third floor is constructed to serve as an interface between the steel and the reinforced concrete structures.

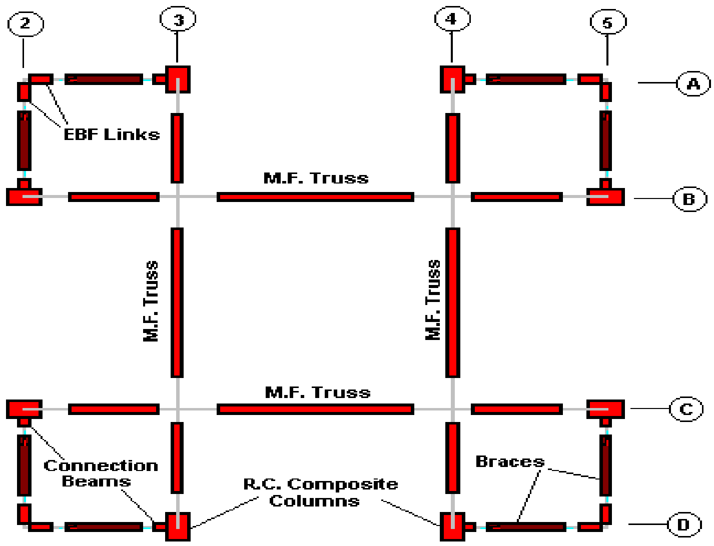


**Figure 1: Typical framing plan**

Wind and earthquake loads were the prime concerns in developing the seismic force-resisting system. The tower's structural performance depended on the system being able to resolve a complex architectural concept without detrimental amounts of torsional displacement. The design process of lateral load-resisting elements was significantly influenced by seismic factors, and a dual seismic-resistant system- eccentrically braced frames (EBFs) and moment resisting frames (MRFs) - was selected for the final version of the structural system.

The primary lateral load-resisting elements above the third floor consists of four steel EBFs, whose link beams are folded at 90 degree to fit the corner shape of the main office floor, resulting in one EBF frame at each corner of the tower. All the EBFs are architecturally exposed, approximately 0.9m outside the facade line, and thus offset from the tower floor diaphragm. These EBFs were designed to carry no floor gravity loads other than own weight. All the corner link beams were detailed to undergo shear yielding during an extreme earthquake event, serving as major earthquake energy dissipation elements for the building. A second set of architectural link beams, their design mainly architecturally influenced, connect the EBFs to the composite column. To minimize complexity of structural behaviour, the architectural link beams were detailed to remain elastic under ultimate loading conditions.

Reinforced concrete space frames transfer seismic forces from the EBFs from the third floor to ground level. Additional two-way reinforced concrete space frames, with columns at 12.6m centre-to-centre, were placed at ground level to adjust the centre of the tower rigidity and control seismic drift from torsional movements. The secondary system of MRFs, provided for structural redundancy, comprises four 26.6m-span trusses moment-connected to the composite columns. The moment connection of the floor truss to the composite column was made by tying both top and bottom chords to the column. The four EBFs frames were therefore coupled with the MRFs trusses through the exterior composite columns.

Because the EBFs are separated from the main tower, the floor diaphragms are not continuously attached to the lateral load-resisting system as in a conventional building. All the diaphragm forces have to be transferred to the EBFs through a set of floor diaphragm link elements. When loads are applied to each floor diaphragm, the forces pass through the floor diaphragm link elements and are then conveyed by the lateral system down to the foundations. This type of diaphragm-force transfer path was fully tested under various load conditions. The most critical load combination identified from the highest level of resultant stresses in the floor diaphragm link element was used for the design and detailing.

The seismic response characteristics of the CEC building is highly dependent on the design of the EBFs' corner link beams which are the major energy dissipation elements for the dual seismic-resisting system. The design of these link beams is based on the criterion that, during an extreme earthquake event, yielding and damage to the system must be limited primarily to these link beams, which will deform inelastically with significant ductility and energy-dissipation capacity. The shear force demand on the corner link beams was first determined from the response spectra analysis, and the corresponding flexural capacity of the link beams was calculated. Once the properties of the corner link beams are defined, the rest of the EBFs elements are sized, with the inclusion of the shear strain hardening effects of the link beam, such that they remain elastic during an earthquake event. For example, the member size of an architectural link beam is determined by increasing its shear yielding capacity at least 1.5 times the shear capacity of the corner link beam. In addition, each architectural link beam is also designed to have a higher value of flexural capacity than that of its shear capacity to eliminate inelastic response under any unforeseeable condition. The detailing requirements for EBFs link beams prescribed in the UBC were strictly followed. Each link-to-brace joint is laterally restrained at both the top and the bottom flanges of the link beam by two TS steel tubes. For the construction of the 3D EBFs, the link beam corner connections and the link beam-to-brace connections are all shop-welded. Field splices are made only for braces where no yielding would be expected.

## SITE CONDITIONS AND TAIWAN BUILDING CODE

The project is in the Taipei basin, which covers some 24 million square meters at around 20m below ground surface. The basin is filled with unconsolidated sediments characterized by a long predominant vibration period. Based on the Taiwan Building Code [6], a local seismic code, the site is in a region of high seismic hazard.

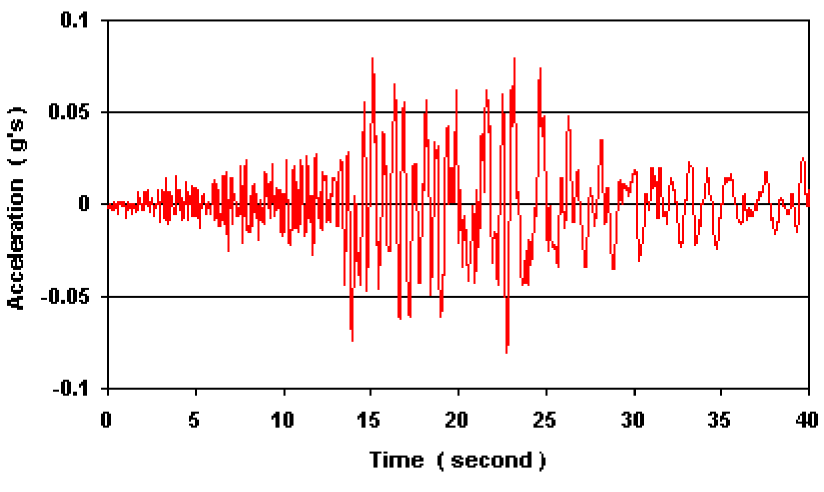


**Figure 2: Recorded ground acceleration in Taipei Basin (CKS4012)**

Moreover, in the Taipei basin, the seismic response of buildings with a fundamental period larger than 1.0 second does not attenuate as rapidly as that in stiff and shallow soil conditions because of the basin effect. Amplification of earthquake ground motions by near-surface geological conditions has been recognized as a major cause of damage in the Taipei basin. The site-response analysis from the past strong-motion records shows that a peak value of response spectra curves always appears at the period of 1.65 second, regardless of the depth of sedimentary deposit. Therefore, the maximum response spectrum value for the Taipei basin extends to the point of T=1.65 second in the Taiwan seismic code.

For the analysis of the CEC building, the processed ground motion records were obtained from the Taiwan Strong Instrumentation Program (TSIP) to represent recorded acceleration time history in Taipei basin. Figure 2 shows a representative recorded absolute acceleration time history with a peak ground acceleration (PGA) of 0.08g. It is, however, noted that the duration of strong ground shaking is fairly long and the ground accelerations are rich with the period domain in the range of 1.0 to 2.0 seconds. The seismic hazard analysis for the Taipei basin, conducted by Loh, Hwang and Shin [5], suggests that the estimate PGA value at Taipei basin is 0.23g for an earthquake event with a return period of 475 years.

Studies indicate that there is a significant difference between the elastic response spectra for buildings in the Taipei basin (by TBC [6] ) and those for the seismic zone 4 of soil type III in California (by UBC 1994 [3] ). For a wide range of medium- to high- rise buildings, the minimum earthquake design base shear for the Taipei basin (by TBC) would be substantially higher than that for similar structures located in seismic zone 4 in California; thus, the seismic force from the TBC was a governing consideration in the design of lateral load-resisting elements.

## INELASTIC ANALYSIS AND SEISMIC PERFORAMNCE

The capacity of a particular building and the demand imposed upon it by a given earthquake ground motion are not independent. One source of this mutual dependence would be evident from the capacity curve itself. The main objective of inelastic analysis is to develop the building capacity curve using static pushover procedures, and to estimate inter-story drifts and plastic rotation demand on shear links, MRFs beams and columns, imposed by earthquake ground motions.

### Modeling Assumptions:

Since the lateral load resisting system is approximately symmetrical with respect to the building configuration in both principal directions, a 2-D structural model was constructed to represent the system characteristics. The 2-D model was developed by condensing one-half of the building lateral system into a plane frame, which consists of two layers. One of them is at one-half of two corners EBFs along grid D, and the other is the secondary MRFs along grid C. The computer program DRAIN-2DX [7] was used for the analysis.

The assumptions made in the analytical model are: (1) concrete composite columns and moment-resisting elements in the trusses are modelled with element type 2 (beam-column), (2) equivalent beam type elements are used to model link-beam yielding in shear, (3) the member yielding strength is defined as the plastic capacity for steel members and as the nominal strength for reinforced concrete and composite members, (4) a 2% strain-hardening for steel elements, and (5) 100% rigid-end offsets at the beam-column joints.

### Non-Linear Static Pushover Analysis:

Various analysis methods, both elastic (linear) and inelastic (nonlinear), are available for the seismic design and evaluation of building. The most basic and simple inelastic analysis method is static pushover procedures, which were developed in the guidelines for the seismic rehabilitation of buildings (ATC-40, 1996 [1] ). The pushover analysis provides a base shear versus roof displacement relationship -system capacity curve, which illustrates the change in stiffness as well as lateral load-carrying capacity of a structure. The change in the slope of the capacity curve suggests yielding of structural elements.

With the code-specified (triangular) load pattern and the first mode-based lateral load pattern, the structure was pushed until its roof drift ratio reaches 1.5% and 2.1%, respectively. Figure 3 shows base shear ratio (V/W) versus roof drift ratio by these two different load patterns. It was found that the ultimate strength of the building is almost 2.7 times the BTS minimum design base shear (Vb=0.12W).

The plots of inter-story drift ratio versus the corresponding total story shear (Total) , the story shear resisted by the eccentrically braced frames (EBFs) , and the story shear resisted by the moment resistant frames (MRFs) are shown in Figure 4 for a typical story level of the building. The code-specified lateral load pattern was used in the plot. Compared with other lateral load patterns, higher mode effects were better represented using the code-specified load pattern. Inelastic behaviour is confined essentially to the EBFs of the dual system for all the stories. It is noted that the EBFs was four times stiffer than the MRFs, which responded elastically until the inter-story drift approached approximately 1.2%.

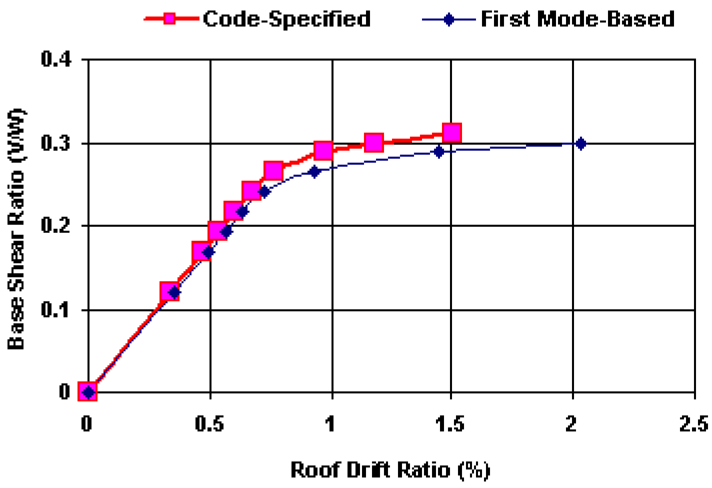


**Figure 3: Base shear ratio vs. roof drift ratio**

Figure 4 also illustrates that the strength of the structure at the individual story level was stable (non-decreasing) following shear yielding in the EBFs links because of the ductility of the EBFs and the significant strength of the MRFs. As mentioned previously, the targeted global drift ratio at the roof level, for the code-specified load pattern, was 1.5% in the pushover analysis. However, the local inter-story drift ratios exceeded 1.5%, and distributed almost uniformly over the height of building. The maximum plastic shear strain of 4.8% radians in the EBFs link at story 6 was obtained in the static pushover analysis.

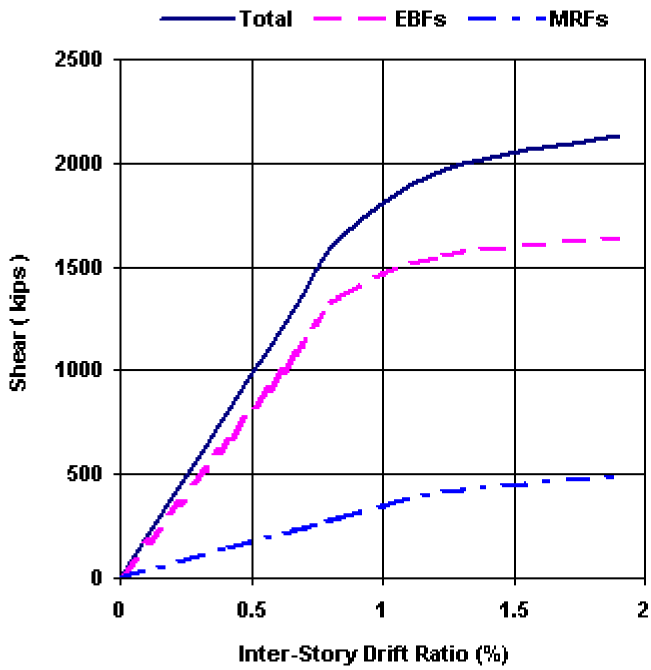


**Figure 4: Story shear vs. inter-story drift ratio**

**Non-Linear Time-History Analysis**

For seismic performance check, the model was further subjected to the recorded earthquake ground motions in Taipei basin. The time-history records of absolute acceleration shown in Figure 2 were scaled up to obtain seismic responses correspondingly to the serviceability earthquake (SE, PGA=0.1g) and the maximum earthquake (ME, PGA=0.34). A damping ratio of 5% was used in the analysis.

The time-history responses indicate that the lateral force profiles were approximately triangular for the serviceability earthquake and approaching to be rectangular for the maximum earthquake. The lateral force distributions for the SE case were similar to those specified by the 1994 UBC [3] or by the 1995 BTS [6] as the design lateral force distributions as well as the fundamental mode shape of the structure.

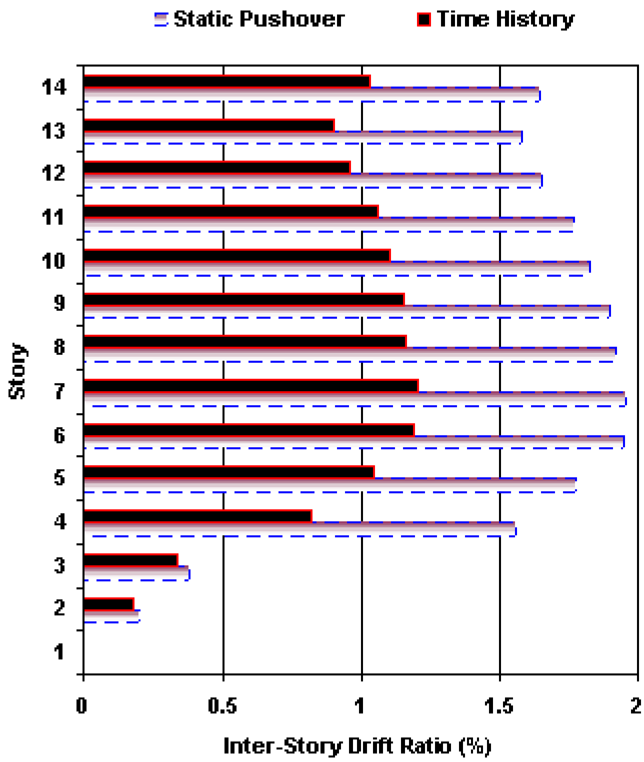


**Figure 5: Envelops of inter-story drift ratio**

The envelopes of inter-story drift ratios from the static pushover analysis with the code-specified load pattern and from the time-history analysis (ME) are plotted in Figure 5. As can be seen, the static pushover drift ratio shape and the time-history analysis results compare reasonably well in all stories.

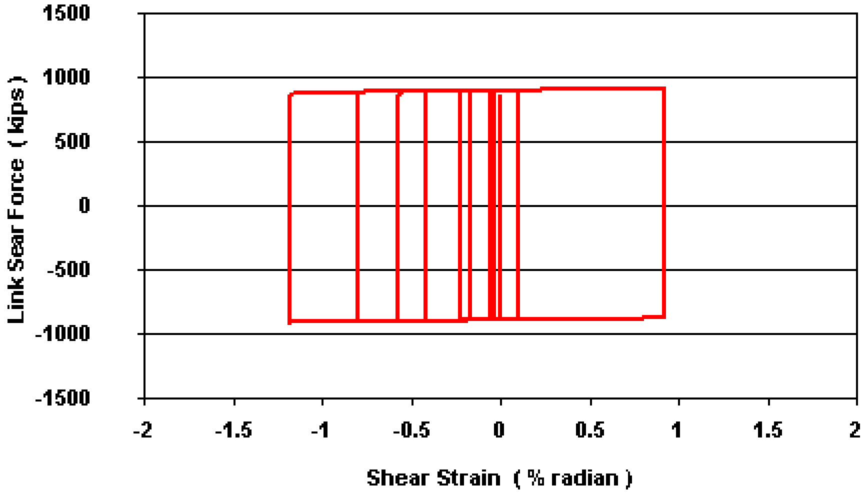


**Figure 6: Shear force vs. shear strain in typical EBFs link**

Figure 6 shows the cyclic loops of shear forces versus shear strains for a typical EBFs link at the maximum earthquake. The maximum plastic hinge distributions from the time-history analysis were also evaluated. It was found that the maximum plastic hinge rotation in the shear link at levels 6 and 8 is less than 1.5% radians and less than 1.0% at other levels. Since the lateral strength of eccentrically braced frames in a dual system is governed by the shear strength of their links, the shear links and their connections to eccentric braces must be detailed to prevent their premature failure. It is usually difficult to predict the failure mode of a shear link in a dual system. It has been established that the local buckling, such as web buckling, leads to the degradation of the shear link's stable hysteretic behaviour. Research results, by Kasai and Popov [4], suggest that a realistic level of deformation in a shear link for the ultimate state is that deformation which associated with the onset of link beam web buckling. For the properly detailed EBFs, the rotation of a link is limited up to 6.0% radians in the 1994 UBC [3].

## CONCLUSIONS

The structural design and detailing of the lateral load-resisting system for the CEC building evolved from an initial scheme based on seismic factors and architect-defined constraints on the structural configuration and drift control, to the development phase based on the seismic hazard analysis of the Taiwan basin, which in turn led to the final design. Both non-linear static pushover and non-linear response time-history analyses were conducted to verify seismic performance of the building.

The dynamic responses of the structure indicate that the dual structural system (EBFs coupled with MRFs) would provide the building with the desired strength and ductility for its seismic performance during both the serviceability earthquake (SE) and the maximum earthquake (ME). The adequate strength and stiffness are the important characteristics for the building to avoid structural and non-structural damages during the serviceability earthquake, and ductile/stable hysteretic behaviours of the building are those such that the structural damages would be predictable during the maximum earthquake.

The analyses focused on inter-storey drifts of the structure and inelastic behaviour of shear links in the EBFs. The strength and deformation compatibility of the EBFs and the MRFs in the dual seismic resistant system were evaluated. The results suggest that the MRFs trusses would develop their potential strengths during the maximum earthquake, which would permit the MRFs to perform its role as the secondary line of earthquake resistance in the dual structural system.

## REFERENCES


1. ATC-40 (1996), "Guidelines for seismic rehabilitation of buildings, Volume 1: seismic evaluation and retrofit of concrete buildings," Applied Technology Council, Redwood City, California.

2. Engelhardt, M.D. and Popov, E.P. (1989), "On design of eccentrically braced frames," Earthquake Spectra, Vol.5, No.3, August, pp495-511.

3. International Conference of Building Officials (1994), Uniform Building Code (UBC), Whittier, California.

4. Kasai, K. and Popov, E.P. (1986), "A study of seismically resistant eccentrically braced steel frame systems," Rep. No. UCB/EERC-86/01, EERC, Univ. of California, Berkeley.

5. Loh,C.H., Hwang, J.Y. and Shin, T.C.(1998), "Observed variation of earthquake motion across a Basin-Taipei City", Earthquake Spectra, Vol.14, No.1, February, pp.115-133.

6. Ministry of the Interior of the Republic of China (1995), Building Technology Standard (BTS), (in Chinese).

7. Parkash, V., Powell, G.H. and Campbell, S. (1993), "DRAIN-2DX, base program description and user guide", Rep. No. UBC/SEMM-9317, Univ. of California, Berkeley, 94pp.

8. Qi, X., Chang, K.L. and Tsai, K. C. (1997), "Seismic design of eccentrically braced space frame," *Journal of Structural Engineering*, Vol.123, No.8, August, pp997-985.