Seismic Assessments of a 34-Story Steel Building Retrofitted with Response Modification Elements

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ABSTRACT :

The original structural design of this case study consisted of five basement floors and a 34-story hotel tower in Kaohsiung, Taiwan. The construction started in 1993 and the erection of the entire steel frame and the pouring of concrete slabs up to the 26th floor were completed before 1996. However, construction of the original hotel was subsequently suspended for 10 years. Recently, this building has been retrofitted and reconstructed for residential purposes. In order to meet more stringent seismic performance requirements unlike the original design, buckling restrained braces (BRBs) and eccentrically braced frames were incorporated into the seismic design of the new residential tower. This paper presents the seismic resisting structural system, seismic design criteria, full-scale test results of one BRB member and the as-built welded moment connections. The paper presents the analytical models and the seismic performance analysis for the original and the re-designed structures.

KEYWORDS seismic retrofit, BRB, EBF, welded moment connection, nonlinear analysis

1. INTRODUCTION

The original structural design was for a hotel tower in Kaohsiung, Taiwan. It is a steel frame building, which consists of five basement floors and a 34-story tower. Construction of the tower started in 1993 and the erection of the entire steel structure and the pouring of concrete slabs up to the 26th floor were completed before 1996. The construction of the original hotel was suspended for 10 years. Recently, this building has been redesigned and re-constructed for residential purposes. The building height remains almost the same, but the floor area in some of the lower floors is reduced while vice versa for the higher floors. The original structural system was no longer able to meet the new seismic force requirements mandated in 2005. Thus, buckling restrained brace (BRB) members and new eccentrically braced frame (EBF) configurations were incorporated. To verify the rotational capacity of the existing welded moment connections, two as-built welded beam-column moment connections were removed from the construction site. A novel stiffening scheme was developed and applied in strengthening one of the connections before tests were carried out to compare the performance of the existing and the stiffened connection details (Weng et al. 2008, Chou et al. 2008). In this paper, the change of seismic force requirements is presented and the new seismic performance requirements for this building are discussed. Test results are presented and analytical models for simulating the experimental responses of the BRB and the stiffened welded beam-to-column connection were examined. Finally, the seismic performance of the structural system and response modification elements were evaluated by conducting a 3-dimensional nonlinear response history analysis of the structure subjected to design base earthquakes in two principal axes.

2. BUILDING STRUCTURE DESIGN

2.1. Original Structural System

The original hotel tower was designed and built as a dual system consisting of steel EBFs and special moment resisting frames (SMRFs) using the Taiwan 1989 Code provisions (Code'89) for seismic force requirements (ABRI 1989). The 1989 and 2005 (ABRI 2005) versions of design force requirements are shown in Fig. 1 in terms of the weight of the structure. The architectural plan of the original hotel tower is similar to those shown in Figure 2. The total height of the original hotel building was 124.7 m. The occupancy importance factor I was 1.25. All columns are of built-up boxes using A572 Grade 50 steel, while the beams are mostly A36. The hotel building fundamental periods computed using the analytical model were 4.27 seconds and 3.74 seconds in the longitudinal and transverse directions respectively. The design seismic base shears based on the Code '89 were 0.033 W (based on I=1.25) for both directions as the vibration period was governed by the empirical formula, $T_{max}=1.4(0.07h^{3/4})=3.66$ seconds.

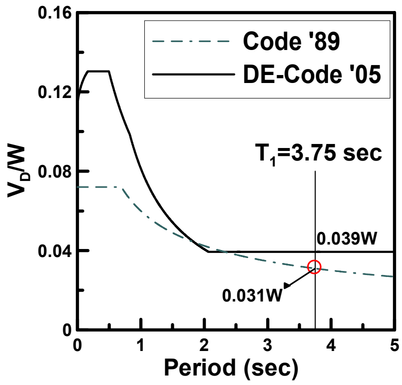

Fig. 1 Design response spectra

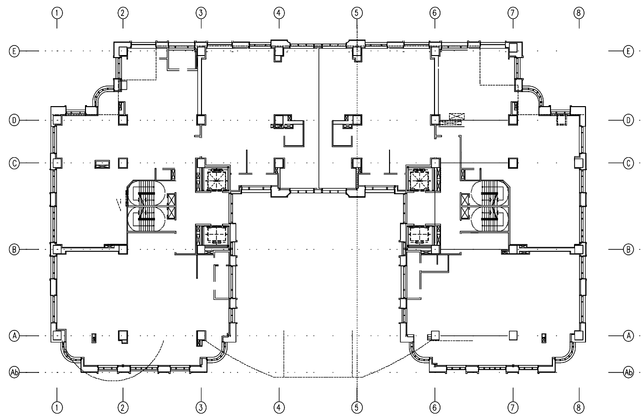

Fig. 2 Architectural plan of the example buildling

2.2. Redesigned Structural System

2.2.1 Design loads

The building has then been redesigned and re-constructed since 2006. The building height remains almost the same, but the floor area in some of the lower floors is reduced while vice versa for the higher floors. Although the total height has been increased slightly from 124.7 m to 128 m, substantial weight of the building has been shifted from the lower floors to the higher floors. Compared to Code'89, the 2005 version of seismic force requirements for buildings has changed rather significantly (Fig. 1). However, the Kaohsiung City Building Department and the structural design review committee agreed with the structural engineers to maintain the use of the same response spectrum suggested in Code '89 for the building seismic retrofit. Nevertheless, the occupancy importance factor I was allowed to change from 1.25 to 1.0. Using the LRFD approach, design load combinations include: (1) 1.4DL, (2) 1.2DL+1.6LL, (3) 1.2DL+0.5LL+1.6WL, (4) 1.2DL+0.5LL+1.0EQ, (5) 0.9DL-1.0EQ, and (6) 0.9DL-1.6WL.The redesigned building fundamental periods computed using the analytical model are 3.75 seconds and 3.60 seconds in the longitudinal and transverse directions respectively. Thus, the design base shear was reduced from 0.033W for the original hotel building to 0.031W (14276 kN) computed for vibration period of 3.75 seconds. The design base shear would have to be increased by 26% from 0.033 W (for original building) to 0.039W if the Code'05 was to be followed (Fig. 1). The design engineers agreed to conduct detailed nonlinear response history analyses (NLRHA). The NLRHA and the performance of the redesigned super-structure will be presented in detail later in this paper.

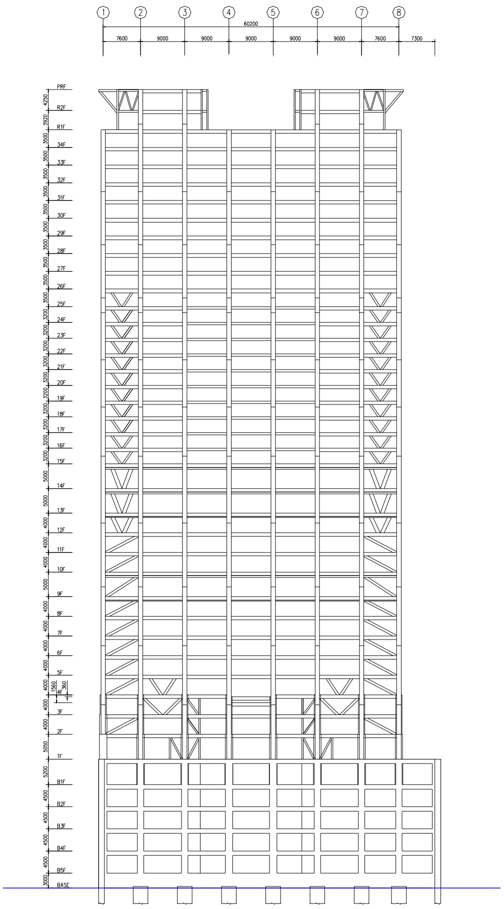

Fig. 3 Longitudinal frame elevation

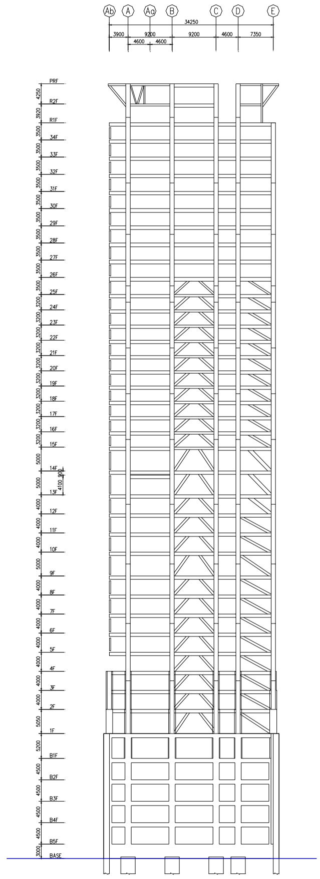

Fig. 4 Transverse frame elevation

2.2.2 ***Upgrading of seismic force resisting system***

Figures 2 and 3 show the typical floor framing plan and the elevation of the redesigned structure. Considering some structural members had deteriorated much due to weathering, it was decided that all steel framing above the 26th floor be removed. New EBFs were chosen for strengthening the 12th to 25th stories whenever the structural and architectural designs allow the installation of the new brace but require accommodating a door opening. In order to enhance the seismic performance, A572 Grade 50 steel BRB elements were added into 1st to 11th stories whenever there was no requirements of providing door opening. Since the pouring of concrete slabs up to the 26th floor was completed before the re-construction, all the existing frame girders using the pre-Northridge type of welded moment connections could not be conveniently removed below the 26th floor. Thus, a new beam-to-column connection stiffening scheme using two steel web side plates were proposed. Test results confirm that the stiffeners effectively make the plastic hinge to form away from the column face and locate at the beam section near the free end of the side plates as illustrated in Fig. 5. The side plates can be conveniently installed in the welded moment connections without removing the concrete slab above the beam flange. More details of the original and redesigned structures, as well as the welded moment connection test results have been documented (Weng et al. 2008).

3. ANALYTICAL MODELS

3.1. Introduction of PISA3D

The Platform of Inelastic Structural Analysis for 3D Systems (PISA3D) is an object-oriented general-purpose computational platform for nonlinear structural analysis (Lin et al. 2008). It provides more than 35 different characteristics of structural elements for simulation of structural responses. In particular, its beam-column element can conveniently simulate the shear yielding or flexural yielding responses of steel wide flange sections. Thus, PISA3D has been applied to carefully model the welded moment connections, BRBs and EBFs in order to investigate the seismic performance of the 34-story steel structure under severe earthquakes.

3.2. Simplified Analytical Approach

3.2.1 Welded beam-to-column moment connections

Without using the rigid end offset feature, the beam-column element flexural stiffness was computed from the node-to-node dimensions. The output of the force responses was also located at the nodal point. To simplify the analytical models and avoid the use of rigid end zones, the yield strength of the beam was modified so that yielding of the beam at the column face can be well represented. The stiffened beam section was not built in the model. However, the plastic hinge was to be formed outside of the stiffeners. As shown in Figure 5, the modified beam (MB model) plastic moment capacity was thus modified accordingly:

$$M_p^{++} = M_p \cdot L_c / L_{sf} \tag{3.1}$$

where M_p^{++} is the modified plastic moment and L_{sf} is the distance from the beam mid-span to the edge of the stiffeners. Figure 6a suggests that the proposed MB model can satisfactorily simulate the experimental response at low-level deformations; while Figure 6b shows that the same MB model can accurately simulate the responses of Specimen 2 at large deformations. Thus, the MB model was adopted for all the welded moment connections in the 3D structural model for the 34-story steel superstructure.

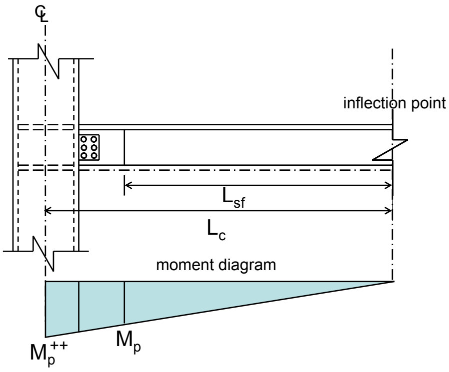

Fig. 5 Illustration of beam moment gradient

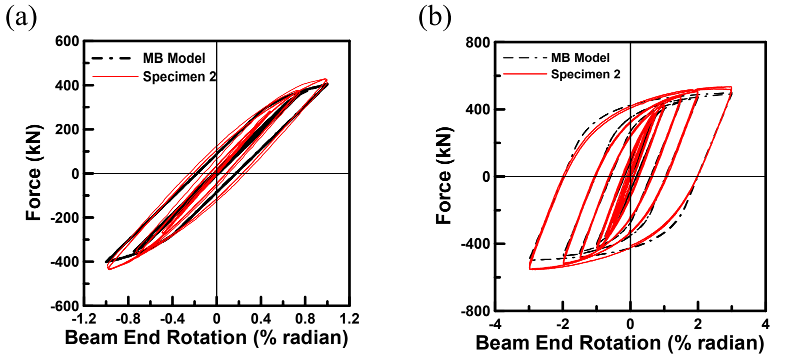

Fig. 6 Test results and analytical model for welded moment connection at (a) low level, (b) high level deformations

3.2.2. EBF shear-link beams

The beam element in PISA3D is able to simulate the shear and/or flexural yielding of the steel wide flange sections. Using the rigid end offset feature for link beam in EBF, the determination of shear yielding or flexural yielding is based on the clear length of the link beam. Therefore, the rigid end offset option was applied so that the shear and flexural strength of the link beam can be correctly incorporated into the determination of shear or flexural yielding. To simplify the analytical model, the panel zone element was not considered at the beam-to-column joint of the link.

3.2.3. Buckling restrained braces

The proposed double core BRB consists of the energy dissipation core yielding segment, the transition region and the core projection (Uang et al. 2004, Tsai el al. 2008a and 2008b). The equivalent axial stiffness, K_e was computed for constructing the PISA3D model using the following equation:

$$K_e = \frac{E \cdot A_j \cdot A_t \cdot A_c}{2A_t \cdot A_c \cdot L_j + 2A_j \cdot A_c \cdot L_t + A_j \cdot A_t \cdot L_c} \tag{3.2}$$

where A_c, A_t, A_j and L_c, L_t, L_j are the cross-sectional area and the length of the energy dissipation core yielding segment, the transition region and the core projection respectively. To examine the quality of the BRBs made by the fabricator, a BRB was arbitrary chosen and tested before the installation of all BRBs. The loading protocol and force versus deformation relationships for this A572 Gr.50 steel BRB specimens are shown with the actual yield capacity (P_y=A_c×$F_{y,actual}$) in Figures 7 and 8. The BRB sustained the standard loading protocol (AISC 2005) before the 19 cycles (not shwn here) of constant-strain (1.5 times the maximum considered earthquake induced strain) cyclic loading were applied. It was evident that the inelastic axial strain of the BRB specimen reached 0.022 (2 times of that associated with the design story drift). In Figure 8, it is demonstrated that the PISA3D analytical model can accurately simulate the cyclic response of the BRB specimen. It is found that the BRB specimen achieved a cumulative plastic deformation (CPD) of 555, greater than the requirement (200 times the yield deformation) before fracture. Figures 9 and 10 show the stiffened moment connections and the BRB members installed at the construction site. All BRB members were modeled using plastic hardening truss elements. The vibration periods of the 1st to 6th mode of the original and the redesigned structures computed by PISA3D are listed in Table 2, respectively.

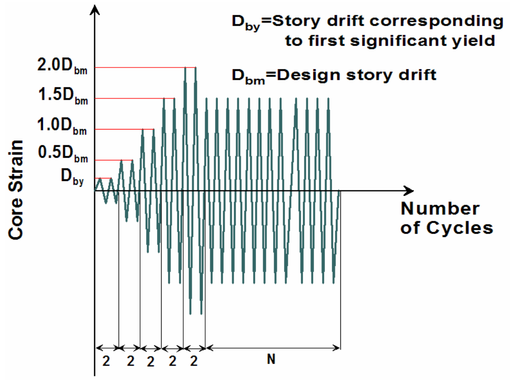

Fig. 7 Standard loading protocol for BRB

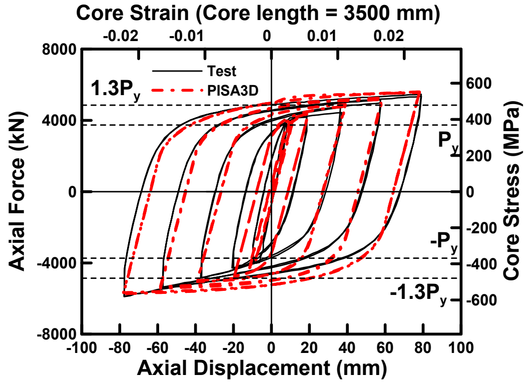

Fig. 8 Analytical and experimental axial force versus deformation response

Fig. 9 Implementation of stiffened welded moment connections in the construction site

Fig. 10 Installation of the BRBs in the construction site

4. SEISMIC RESPONSE ANALYSIS USING SYNTHETIC GROUND MOTIONS

4.1.Ground motions

The phase angles of three historical ground motion records: KAUEW, KAUNS, and KAUPNS, were selected for the construction of the synthetic earthquake ground accelerations (Figure 11). These three records were recorded in Kaohsiung during a recent earthquake that struck at the southern coast of Taiwan on December 26th in 2006 (NCDR and NCREE 2006). Figure 12 shows the acceleration time history of the three synthetic earthquakes, denoted as AKAUEW (EQ1), AKAUNS (EQ2) and AKAUPNS (EQ3). Figure 13 confirms that the response spectra of the three synthetic ground motions are compatible with the elastic design spectrum suggested by Code ’05 for a MCE-level (PGA=0.32g) earthquake.

4.2. Seismic performance evaluation

4.2.1.Nonlinear response history analysis (NLRHA) and incremental dynamic analysis (IDA)

The NLRHA was conducted for both the DE-level (PGA=0.29g) and the MCE-level (PGA=0.32g) earthquakes. However, it was found that the responses of the superstructure were similar under the excitation of these two levels of earthquake. Only the key responses of the original and the redesigned structures under the MCE-level earthquakes are presented. The peak story displacements, peak inter-story drifts, and the peak story shears under the application of these three synthetic earthquakes of MCE-level are shown in Figure 14. It shows the peak roof displacement reached 1.12 m and 1.03 m for the original and redesigned structures, respectively. Figure 14 also shows peak inter-story drifts are reduced from the 1st to 11th floors but increased from the 26th to 34th floors after retrofitting. This could be because the floor weight in some lower floors was reduced while that of the higher floors was increased. Peak inter-story drifts of the original and the redesigned structures reached 0.012 and 0.014 radians, respectively.

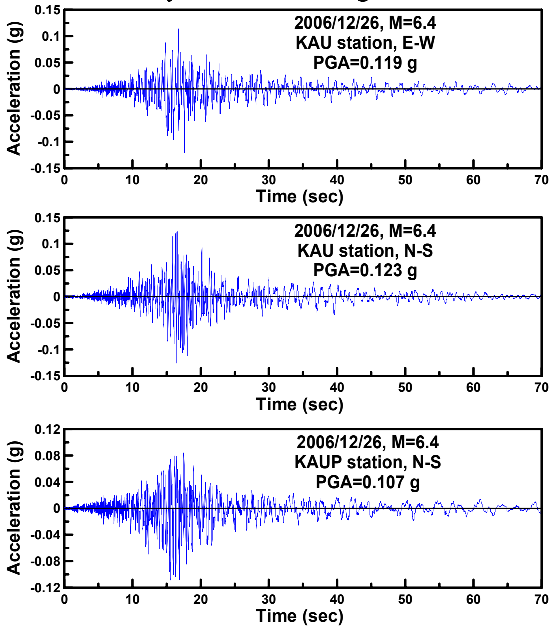

Fig. 11 Historical ground motion records

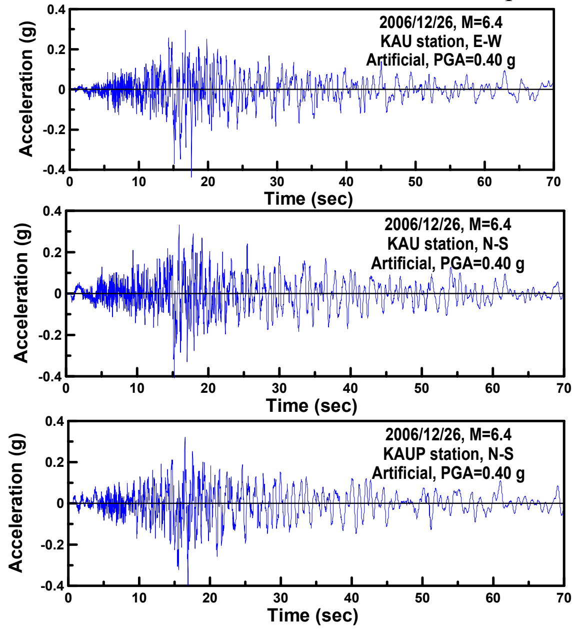

Fig. 12 Synthetic ground motion records

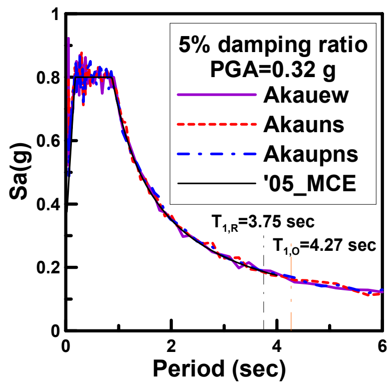

Fig. 13 Confirmation of spectral compatible ground motions

Nevertheless, the peak inter-story drift demand (0.014 radian) under the MCE-level excitation was still small enough to meet the Life-Safety performance criterion (0.015 radian) for new steel braced frames suggested in FEMA-450 (BSSC 2003). Figure 14 also shows that almost all the peak story shears were increased but only slightly after the structure was retrofitted. The incremental dynamic analysis, IDA (Vamvasikos and Cornell 2003) was conducted using the same three earthquakes to compare the capacities between the original and the redesigned structures. The resulting roof drift versus the base shear to building weight ratio relationships are shown in the bottom right of Figure 14. It can be found that the lateral stiffness and the strength of the original structure are smaller than those of the redesigned. Although the responses of the original structure were not significantly reduced after retrofitting, the owner and the design engineer of the building decided to add response modification elements into the re-construction. It is because that the pre-Northridge type of welded moment connection adopted in the original structure may not be able to reliably provide sufficient deformational capacities, especially when the original structural frame was exposed to weathering for more than 10 years.

Under the MCE-level EQ3 earthquake applied in the longitudinal direction, Figures 15a, 15b and 15c show the corresponding hysteretic responses of some specific structural members where nonlinear responses are more pronounced than elsewhere: BRB member at 8th floor, shear-link beam at 21st floor and welded moment connection at 8th floor respectively. The stated BRB peak axial core strain demand was about 0.077 (Fig. 15a), and the peak total beam end flexural rotation demand reached 0.01 radian (Fig. 15b). Figure 15c shows that the link beam total shear rotation demands of the original and redesigned buildings reached 0.065 and 0.033 radians respectively. Both are smaller than the 0.08 radian recommended for the shear link (AISC 2005). Clearly, some of the EBF link beam rotational demand in the original structure has been significantly reduced. The analyses also suggest that the rotational demand imposed on the welded moment connection is only about 0.01 radian, much smaller than the rotational capacity (0.04 radian) found in the test of the retrofitted connection.

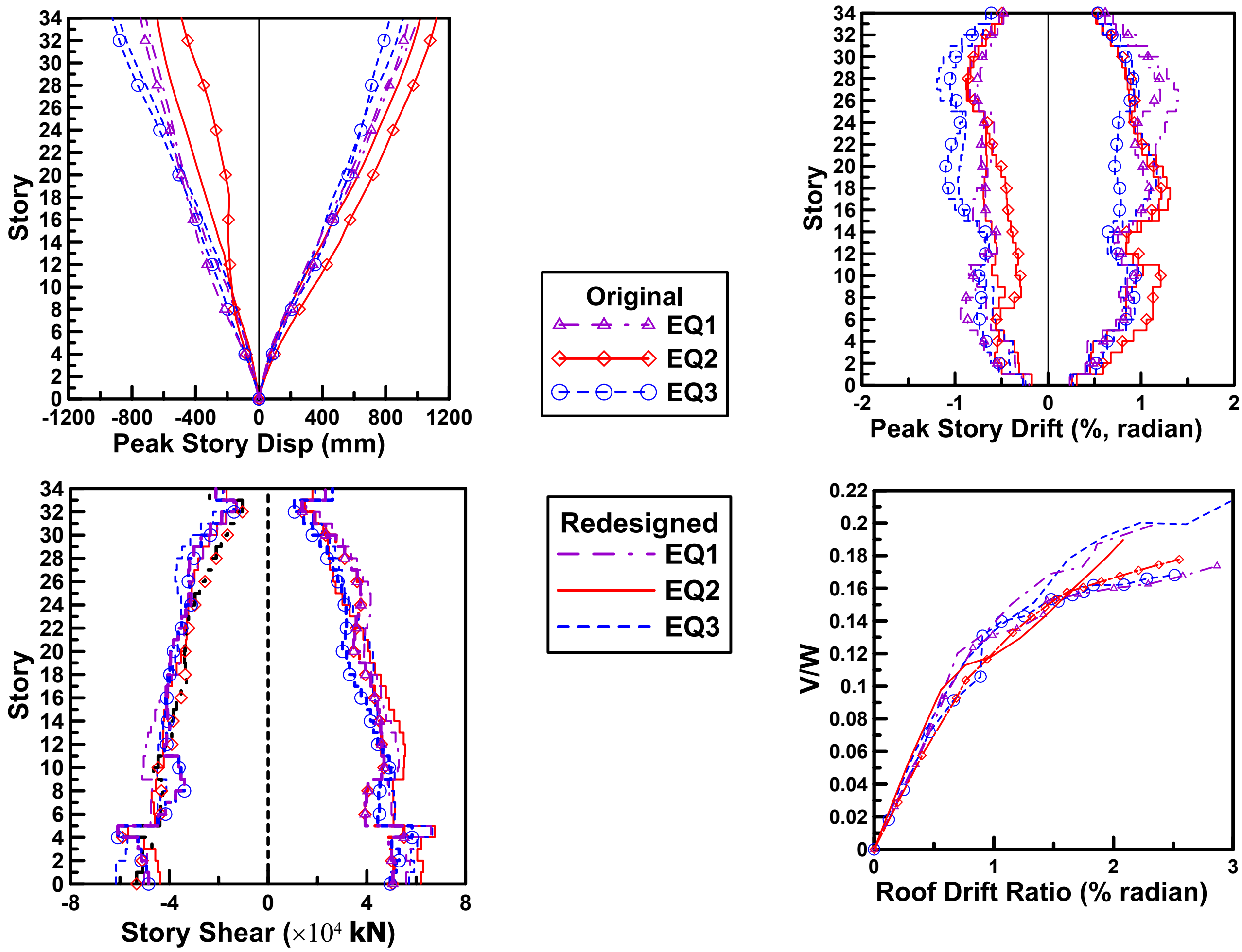

Fig. 14 Comparison of peak seismic responses in the longitudinal direction between the original and redesigned structures induced by synthetic ground motion records: (top left) peak story displacement; (top right) peak inter-story drift; (bottom left) peak story shear; (bottom right) the capacity curves of the redesigned structure obtained from IDA.

5. CONCLUSIONS

From the experimental and analytical studies, the following conclusions can be drawn:

(a) The proposed beam-to-column connection stiffening scheme using two steel web side plates can be conveniently installed in the welded moment connections without removing the existing concrete slab above the beam flange;

(b) Test confirms that the proposed side-plate stiffening scheme is effective in preventing the fracture occurred in the beam flange. The stiffened welded moment connection possesses a larger rotational capacity than that in the original pre-Northridge type connection. A separate study (Chou et al. 2008) has also confirmed its effectiveness and the design method using cyclic tests and finite element analysis for nine additional specimens;

(c) The proposed analytical models are found to be accurate in simulating the experimental responses of BRB member, and welded moment connections before and after stiffening;

(d) Nonlinear dynamic analyses confirmed that the rotational demand on the EBF link beams has been significantly reduced by adopting the response modification elements in the redesigned structure;

(e) Nonlinear response history analyses show that almost all the story shear demands were increased only very slightly after the structure was retrofitted. Analytical results indicate that the ratios between the brace shear and story shear along the building height range from 0.1 to 0.9;

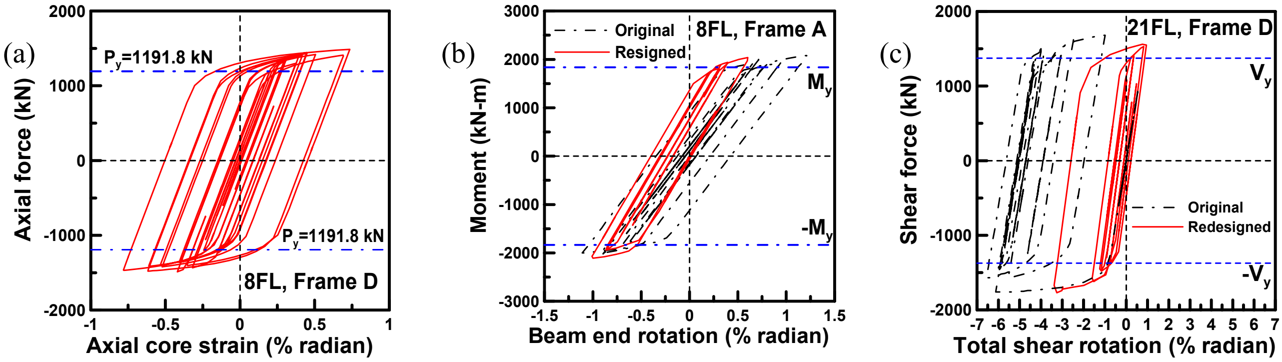

Fig. 15 Nonlinear seismic responses: (a) BRB member ; (b) welded moment connection and (c) shear-link beam induced by the EQ3 acceleration record for the MCE-Level (PGA=0.32g).

REFERENCES

AISC (2005) *Seismic Provisions for Structural Steel Buildings*, Chicago, IL, 2005.

ABRI (1989) *Recommended Provisions for Building Seismic Regulations*, Taipei, 1989. (in Chinese)

ABRI (2005) *Recommended Provisions for Building Seismic Regulations*, Taipei, 2005. (in Chinese)

BSSC (2003) NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, FEMA 450, Federal Emergency Management Agency: Washington, DC

Chou, CC, Tsai, KC, Wang, YY, Jao, CK. (2008), "Seismic performance of steel site plate moment connections", *14th World Conference on Earthquake Engineering*, Beijing.

Lin, B.Z., Chuang, M.C. and Tsai, K.C. (2008), "Object-oriented Development and Application of a Nonlinear Structural Analysis Framework", *Advances in Engineering Software*, (accepted)

NCDR and NCREE (2006) The Preliminary Reconnaissance Report of 2006 Hengtsun Earthquake, December 2006. (in Chinese)

Tsai, K.C., Hsiao, P.C., Weng, Y.T., Lin, M.L., Lin, K.C., Chen, C.H., Lai, J.W. and Lin, S.L. (2008a), "Pseudo-dynamic Tests of a Full-scale CFT/BRB Frame. Part 1: Specimen Design, Experiment and Analysis, Earthquake Engng Struct. Dyn., 37:1081–1098

Tsai, K.C., Hsiao, P.C. (2008b), "Pseudo-dynamic Tests of a Full-scale CFT/BRB Frame. Part 2: Seismic Performance of Buckling Restrained Braces and Connections, Earthquake Engng Struct. Dyn., 37:1099–1115

Uang, CM, Nakashima, M and Tsai, KC. (2004), "Research and Applications of Buckling Restrained Braced Frames", *International Journal of Steel Structures*, 4/2004.

Vamvatsikos D. and Cornell A. (2002), "Incremental Dynamic Analysis", *Earthquake Engineering and Structural Dynamics* 2002; **31**(3): 491-514.

Weng, YT, Tsai, KC, Chen, PC, Chou, CC, Chan YR, Jhuang, SJ, and Wang YY (2008), "Seismic Performance Evaluation of a 34-Story Steel Building Retrofitted with Response Modification Elements", *Earthquake Engineering and Structural Dynamic*, (in review)