Case Study on Seismic Retrofit Design and Performance Evaluation of A 34-Story Steel Building

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Abstract

The construction of a five-floor basement and 34-story steel building was started in 1993. The erection of the steel structure and the pouring of concrete slabs up to the 26th floor were completed in 1996. However, due to the financial difficulty of the hotel developer, the construction of the original structure has been suspended for more than 10 years. Recently, this building is being retrofitted and re-constructed for residential purposes. In this paper, the change of seismic force requirements for buildings in Taiwan after the 1999 Chi-Chi Earthquake is introduced first, the new seismic performance requirement for this building is then discussed. In order to meet a more stringent seismic performance requirement than the original design, buckling restrained braces (BRB) and eccentrically braced frames (EBF) with shear links were incorporated into the seismic design of the new residential tower. In addition, two as-built welded beam-column moment connections were removed from the existing construction site. A novel stiffening scheme was applied in strengthening one of the connections before the tests for verification of the rotational capacity. This paper presents the simplified methods of simulating the experimental responses of the beam-to-column connections. The paper concludes with the nonlinear seismic resisting performance of the building model subjected to earthquakes in two principal axes.

Keywords: Seismic performance upgrading, BRB, EBF, shear link, nonlinear analysis.

1. INTRODUCTION

A five basement floors and one 34-story, 128m height steel structure is being retrofitted for an upscale residential tower in Kuohsiung using various types of seismic response modification elements. The construction of the tower was started in 1993 and the entire steel structure and the pouring of concrete slabs up to the 24^{th} floor were completed in 1996. However, due to the financial difficulty of the developer, the construction has been suspended for more than 10 years. Recently, this building is being re-constructed for residential purposes. The building height remains pretty much about the same, but the floor area in some of the lower floors is reduced while the higher floors' area has been increased. In this paper, the change of seismic force requirements for buildings in Taiwan after the 1999 Chi-Chi Earthquake is introduced first, then the new seismic performance requirement for this building is discussed. Moreover, in order to meet a more stringent seismic performance requirement than the original design, buckling restrained braces (BRB) and eccentrically braced frames (EBF) with shear links were incorporated into the seismic design of the re-constructed residential tower. In addition, two as-built welded beam-column moment connections were removed from the existing construction site. A novel stiffening scheme was applied in strengthening one of the connections before the tests for verification of the rotational capacity. This paper compares the test results of these two connections and discusses the seismic performance of the building using the 3-dimensional nonlinear dynamic response analysis of the structure subjected to design base earthquakes in two principal axes.

2. BUILDING DESCRIPTION AND DESIGN DETAILS

This building was first designed and built as a dual system consisting of steel EBF and special moment resisting frame (SMRF) using the 1989 provisions on seismic force requirements. Originally it was to be served as a hotel building before the construction has been suspended since 1996. However, the seismic force requirements for buildings in Taiwan were substantially changed in 1997 and 2005, denoted as Code '97 and Code '05 respectively. The local city building department and the structural design review committee agreed with the structural engineer to maintain the use of 1989 Taiwan seismic force requirements (denoted as Code '89) for building's seismic retrofit and reconstruction for residential purposes. The building's fundamental period in the transverse direction is about 3.73 sec. The design seismic base shears computed from Code '89 and Code '05 are 0.031W and 0.039W respectively. Considering some structural members had been deteriorated so much after the construction was suspended, it was decided to remove all the steel framing above the 26^{th} floor. In order to enhance the seismic performance of this building, BRB elements and EBF systems were added in the 1^{st}–to-11^{th} and 12^{th}-25^{th} stories respectively. In addition, stiffened beam-to-column connection details using two steel web side plates were tested first then implemented in all the welded beam-to-column connections below the 26^{th} story [1].

3. CYCLIC TESTS ON BEAM-TO-COLUMN CONNECTIONS

The proposed stiffening scheme was applied in the beam-to-column moment connections to ensure that the rotational capacity of welded moment connections are sufficient to meet the most modern seismic steel building design provisions. Figure 1 shows the details of the proposed strengthening scheme: two full height steel web plates (t=20mm) were welded to the column face and to the edges of the top and bottom beam flanges at the beam end. Two as-built welded beam-column moment connection subassemblies were cut from the 33rd floor. These two specimens are designated as Specimen 1 and 2, which represent the steel beam with or without the proposed stiffening scheme, respectively. The retrofitting effects of the proposed stiffening scheme were investigated. Figure 2 shows the cyclic loading test results of Specimen 1 and 2. Test results are introduced in the following.

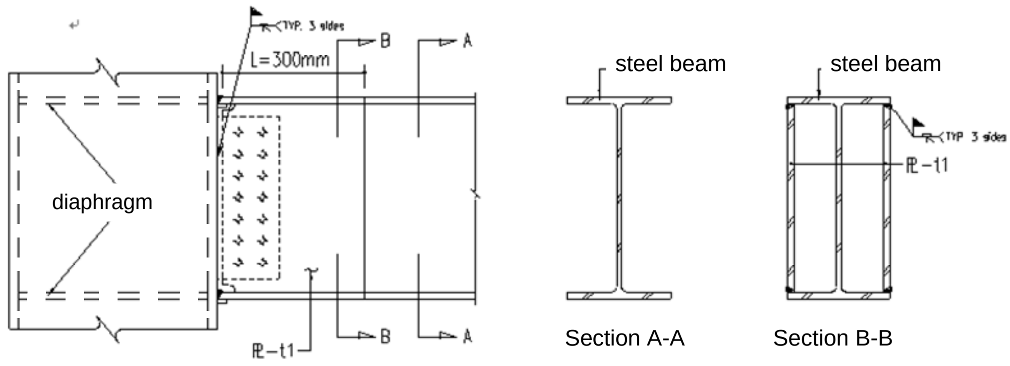


Figure 1 The strengthening details of the welded moment connections

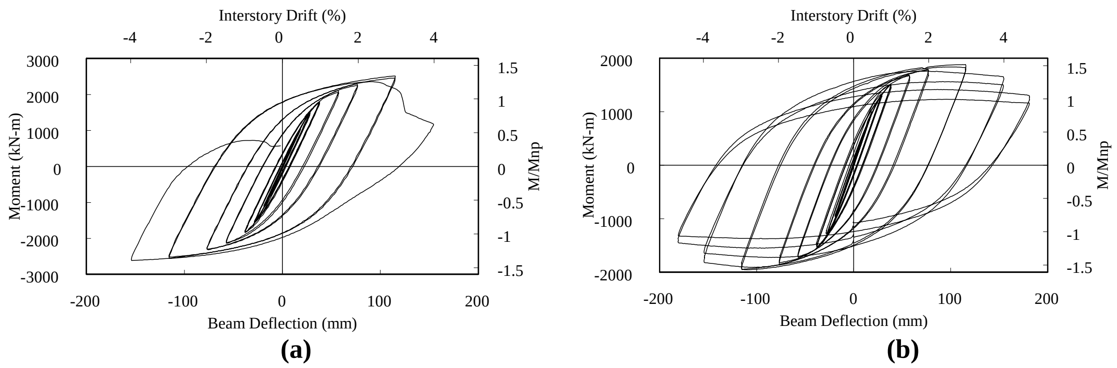


Figure 2 The cyclic test results: (a) Specimen 1 (b) Specimen 2

4. ANALYTICAL MODEL

The Platform of Inelastic Structural Analysis for 3D Systems (PISA3D), developed in National Center for Research on Earthquake (NCREE), Taiwan, is an object-oriented general-purpose computational platform for structural analysis [2]. Users can conveniently build 3D numerical models using the material and element libraries provided in PISA3D. Currently, there are more than 7 types of yielding rules and 5 types of nonlinear elements. Thus, it provides more than 35 different characteristics of structural elements for simulation of structural responses. In this paper, PISA3D has been applied to model BRBs and EBFs in order to investigate the seismic performance of the 34-story steel structure.

Without using the rigid end offset option, the beam-column element stiffness is computed from the node-to-node distance. The output of the force responses is also at the nodal point. In order to simplify the analytical model, the yield strength of the beam is modified so that the yielding of the beam at the column face can be well represented. As shown in Figure 3, the input moment capacity is modified as the following equation:

$$M_p^+ = M_p \times L_c / L_f \tag{1}$$

Where M_p is the plastic moment capacity of the bare steel beam, M_p^+ is the modified input moment capacity, L_c is the distance from the inflection point to the center of the column, L_f is the distance from the inflection point to the column face. Figure 4 compares the simulation and the test results of the Specimen 1. Obviously, the proposed modified method applied on the strain hardening beam-column element model can well simulate the responses of the specimen. The same technique is found equally effective to apply in modeling the stiffened steel beam Specimen 2. However, in order to save computation time, bi-linear material, rather than the strain hardening material model shown in Fig. 4, was adopted for beam-column elements in the preliminary analyses.

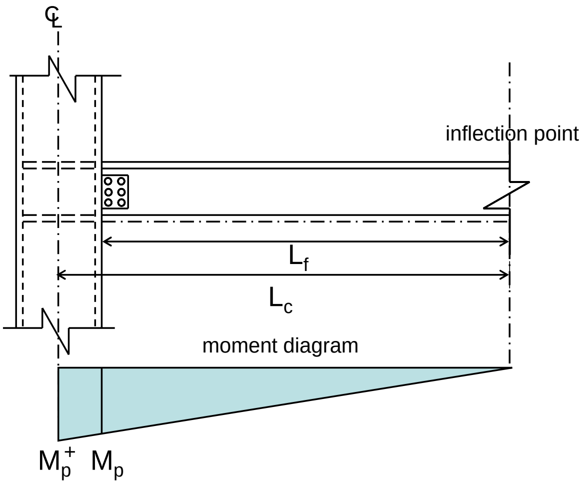


Figure 3 Illustration of the moment gradient for flexural capacity modification

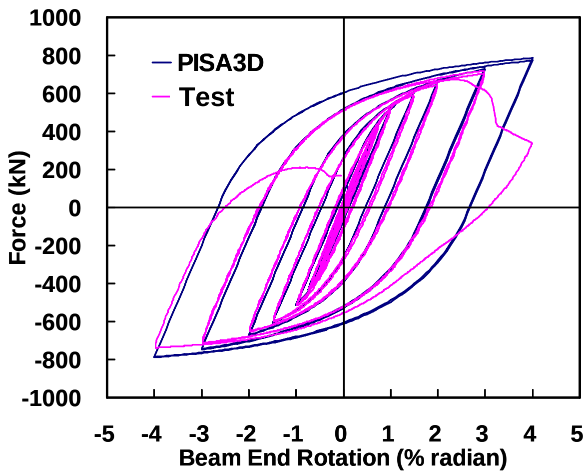


Figure 4 Comparison of analytical and test results (Specimen 1)

Likewise, without using the rigid end offset feature for link beam in EBF, the determination of shear yielding or flexural yielding is based on the node-to-node distance of the link beam. Thus, in the simplified model, the shear and flexural strength of the link beam are also modified. As shown in Figure 5, in order to ensure the shear yielding of the link beam can be satisfactorily detected, it is necessary to modify the shear link yield strength as follows:

$$V_{yc} = V_y \times L_L / L_M = \tau_y \times (A_v \times L_L / L_M) \tag{2}$$

Where V_{yc} is the modified shear capacity (for input) of the link beam, V_y is the nominal plastic shear capacity of the link, τ_y is shear yield stress of steel, A_v is the shear area of the link, L_L is the distance from the work point in the EBF to the face of the column, L_M is the distance from the work point to the center of the column. Reverse modification needs to be applied in order to get the force-deformation relationships of the link. In the preliminary, all the link beam elements were built using the strain hardening material.

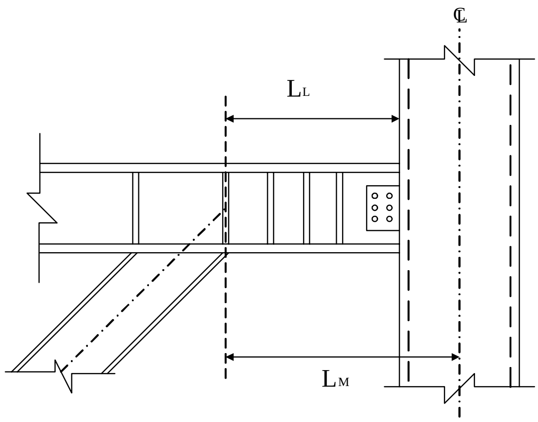


Figure 5 Illustration of the input shear strength modification

The buckling restrained bracing (BRB) consists of the energy dissipation core segment, the transition region and the end-joint segment [3]. The equivalent axial stiffness K_e is computed first before constructing the PISA3D model using the following equation:

$$K_e = \frac{E \cdot A_j \cdot A_t \cdot A_c}{2A_t \cdot A_c \cdot L_j + 2A_j \cdot A_c \cdot L_t + A_j \cdot A_t \cdot L_c} \tag{3}$$

Where A_c, A_t, A_j and L_c, L_t, L_j are the area and the length of the energy dissipation core segment, the transition region and the end-joint segment, respectively. However, the plastic response develops only at the energy dissipation core segment, hence A_c is required to represent the cross-sectional area of the bracing in the analytical model. In additional, the steel material's Young's modulus is modified in order to properly model the elastic axial stiffness using the following equation:

$$E = \frac{K_e \cdot L_{wp}}{A_c} \tag{4}$$

Where L_{wp} is the distance from the work point to the work point of the bracing. It is the same distance from the node to node in PISA3D model. Likewise, reverse modification needs to be applied in order to investigate the force-deformation responses of the bracing. In the preliminary analyses, all the BRB members were built using strain hardening material for truss elements. Figure 6 and Figure 7 shows the isometric 3D view of analytical model and the 1^{st}-3^{rd} mode shapes obtained from PISA3D. The vibration periods of the 1^{st} to 6^{th} modes computed by a commercial software [4] and PISA3D respectively are listed in Table 1.

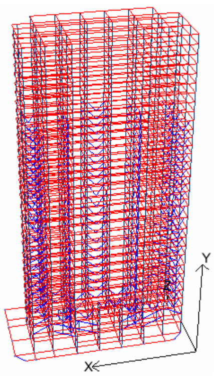


Figure 6 The isometric view of the analytical model

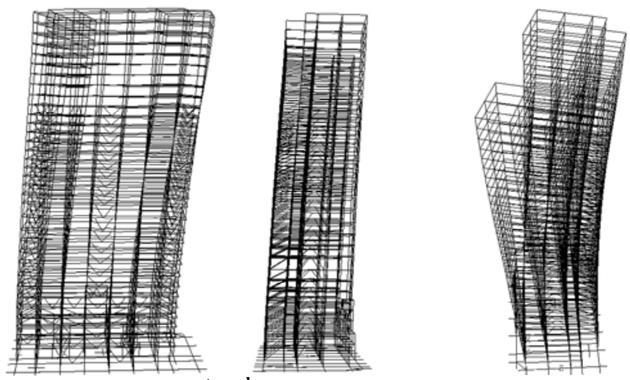


Figure 7 The 1^{st}-3^{rd} mode shape of the analytical model

Table 1 Comparison of period of the 1^{st}-to-6^{th} modes between ETABS and PISA3D

	ETABS (sec.)	PISA3D (sec.)
1^{st} Mode	3.75	3.74
2^{nd} Mode	3.60	3.59
3^{rd} Mode	3.20	3.21
4^{th} Mode	1.21	1.21
5^{th} Mode	1.18	1.18
6^{th} Mode	1.11	1.08

5. SEISMIC RESPONSE ANALYSIS

The phase angles of three historical ground motion records KAUEW, KAUPEW, and KAUPNS were selected for the construction of artificial earthquake time histories in which its acceleration response spectra are compatible with the design response spectrum. These records were recorded in Kaohsiung during a recent earthquake that struck the southern coast of Taiwan on December 26^{th} in 2006. Figure 8 shows the response spectra of these three artificial ground motions, quite compatible with the elastic design spectrum suggested by the Code ’05 for MCE-level (PGA=0.32g). Figure 8 displays the elastic spectra of each generated earthquake ground motions, denoted as

AKAUEW (EQ1), AKAUNS (EQ2) and AKAUPNS (EQ3). Figure 9 shows the acceleration time history of the three artificial earthquakes.

The peak story displacement, peak inter-story drift ratios, and the peak story shear under the application of the noted three earthquakes for MCE-level are shown in Figure 10. Peak inter-story drift ratio reached to 1.12% radians, and peak roof displacement reached 1.0 meter. The peak inter-story drift demand under the MCE-level excitation was small enough and even met the Life-Safety performance criterion (2.5% radians) for existing buildings suggested in FEMA-356 [5]. Figure 11 illustrates the plastic hinge distributions in one of the lateral force resisting frames of the building in the longitudinal direction under the MCE-level excitations. The peak plastic beam end flexural rotation reached 0.65% radians, and the peak plastic link beam shear deformation reached 6.5% radians.

In order to estimate the capacity curves and deformation demand under a specific hazard level of this building, incremental dynamic analysis (IDA) proposed by Vamvatsikos and Cornell (2002) [6] was conducted. Figure 12 shows the capacity curves by scaling EQ1 through EQ3 records to various levels of intensity. According to the Code’05, the design earthquake force is 0.039W, the corresponding roof drift ratio is about 0.25% radian. It appears reasonable to assume that the under the Code’05 lateral force, the structural system is well likely to remain elastic.

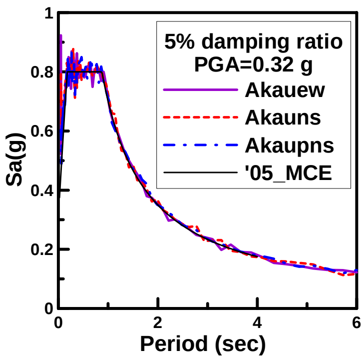


Figure 8 The design and compatible elastic spectra

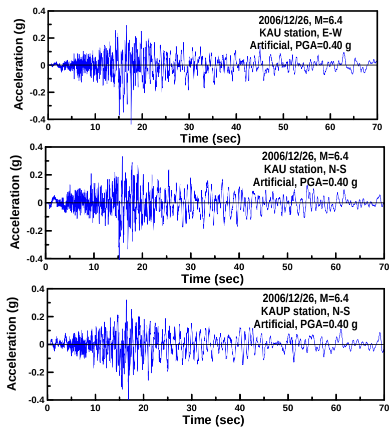
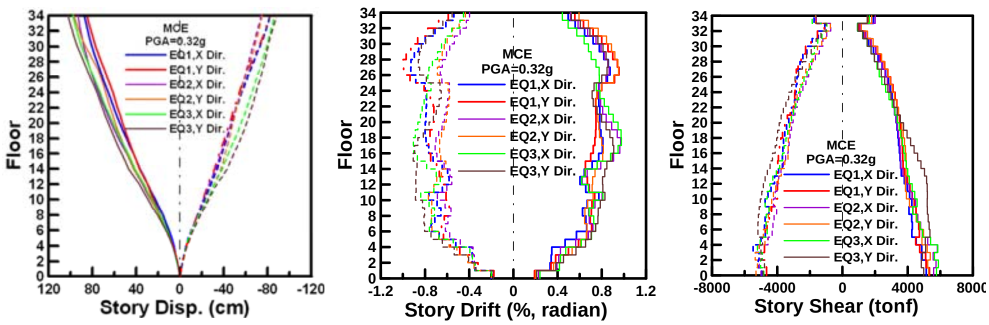


Figure 9 The acceleration time history records of artificial earthquakes



(a) (b) (c)

Figure 10 The peak response of the model: (a) story displacement (b) story drift (c) story shear

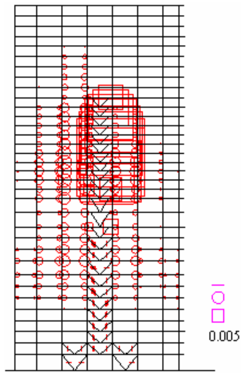


Figure 11 The illustration of plastic hinge on frame D (MCE-level)

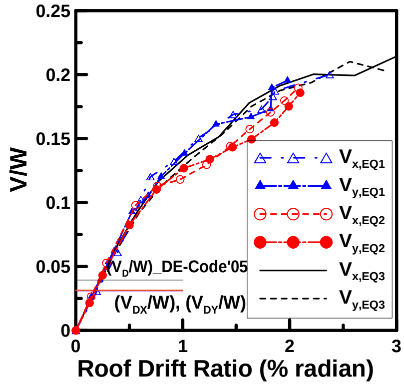


Figure 12 The capacity curves obtained from IDA

6. SUMMARY and CONCLUSIONS

The analytical model for this 34-story steel building consists of three primary laterteral force resisting systems, including SMRF, BRBF, and EBF systems. Based on the stated experimental and analytical studies, conclusions of can be drawn as follows:

(a) Nonlinear dynamic analyses were employed in the case study of the noted building to identify nonlinear dynamic characteristics, including yielding mechanisms, deformational demands, and detailing requirements.

(b) A novel stiffening scheme for existing welded beam-to-column moment connection was verified by full-scale cyclic loading tests.

(c) Analytical results suggest that the seismic retrofit design of the noted building is effective. The deformational demands of the proposed seismic force resisting elements are smaller than those found in the laboratory tests.

7. REFERENCE

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