AN INNOVATIVE DESIGN STRATEGY FOR SEISMIC RETROFITTING OF A HIGH-RISE STEEL BUILDING IN KAOSHIUNG

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ABSTRACT:

After the devastation and damage caused by the Chi-Chi Earthquake in 1991, Taiwan initiated progressive seismic building code provisions to strengthen design requirements for building design and construction. In response to the enhanced seismic design requirements, this case study elaborates on some innovative approaches to meet the stringent building code provisions, while addressing some unique experiments and analytical results applied for the retrofit of the once abandoned high-rise steel buildings located in Kaoshiung. The incomplete high-rise steel building in question was abandoned for various reasons. The main structural members were exposed to harsh weather and some had suffered from erosion. In order to meet the newly adopted seismic code requirements, it becomes a challenging task to complete this construction when building occupancy use also changes. A newly proposed beam-column connection was tested in order to prove its intended strength. Addition of buckling retrained braces provided damping to enhance this building's seismic performance. The structural system was consequently changed to an eccentrically braced frame system to meet the stringent building draft requirement. Due to the obvious complexity of the structural performance as a result of these additions and revisions, a nonlinear structural analytical study was conducted to investigate the global behavior of the newly revised building. This case study aims to illustrate how innovative design strategies and construction techniques lead to a successful building retrofit project.

KEYWORDS: buckling restrained braces, damping, eccentrically braced frame system, retrofit

1. INTRODUCTION

When the building was originally constructed, it was planned to be a 34-story steel frame hotel above ground with a 5-floor underground parking garage in Kaoshiung. The construction began in June 1993 and stopped short of completion three years later in 1996. The steel building frame was completed on the top roof while the steel-composite floor construction stopped on 25th floor. It was retrofitted to be a modern residential building. The renewed story height now is 3.5 M instead of 3.2 M per floor.

The steel building frame from 25th to the roof was completed replaced while the lower composite steel floors remain. The 12th floor to 25th floor maintains the original eccentrically braced frame (EBF) while the ground floor to 11th floor has the buckling restrained braces (BRB) as energy dissipation devices for both earthquakes and wind. (Shuhaibar, C., 2000) Additionally, the columns from the ground floor to the third floor are box steel members with composite concrete. The basement was built with reinforced concrete and the mat foundation has piles driven up to 41.5 M with the pile diameter around 2.0 to 2.4 M.

1.1. Site Investigation

The site investigation included determining: (1) whether the tolerance for error of the constructed structural members is acceptable when compared to the original design; (2) the erosion status of the main structural members and the connections; (3) if the basement reinforced concrete has cracks and erosion, and (4) existing structural integrity of the steel, bolts, concrete, soil, and steel reinforced concrete by testing various samples.

1.2. D-Class erosion test for beam-column connection

The beam-column erosion test results were based on the SIS-05-5900 standard. Six of those classified as D-class eroded steel members with sizes of 300 mm x 100mm were tested by Taiwan SGS test company for the tension-compression, erosion and chemistry elements that are essential to safety. According to the test results, they summarized that: (1) the erosion thickness was between 0.33 ~ 0.55 mm; (2) the coupon test indicated that the tensile strength, yielding strength and strain met all ASTM steel standards; and (3) elements such as C, Si, S, P and Mn dictated in the beam-column samples all met standards for safety.

2. STRUCTURAL RETROFIT STRATEGY AND ANALYSIS PROCEDURES

The original main structural members included both moment resisting frames and eccentrically braced frames and were designed based on the 1989 Taiwan Building Code. The designed seismic base shear force was equal to 0.031W, where W is the total weight of the building. Since 1989, the Taiwan Building Code (TBC) has been updated few times. See Figure 1 for the different results of base shear force based on various versions of the TBC. According to the current TBC2006, the seismic force for this building would be 0.039W, which is about a 25% increase compared to TBC1989. Therefore, this project seeks nonlinear structural analysis and dynamic analysis to ensure the enhanced structural performance for earthquakes.

2.1 Taiwan Building Code 2006

The Ground Motion Parameters of site mapped design and maximum spectral acceleration parameters at the short period and 1-second period for Kaoshiung City can be directly obtained from Table 2-1 of TBC2006 and site coefficients from Table2-2. The following parameters can be obtained once the site seismic information is determined. $S_S^D, S_1^D, F_V^D, S_S^M, S_1^M, F_V^M$. Using the following equations, one can get the design parameters.

$$S_{DS} = F_a^D x S_S^D, S_{MS} = F_a^M x S_S^M, S_{D1} = F_V^D x S_1^D, S_{M1} = F_V^M x S_1^M,\ T_0^D = S_{D1}/S_{DS},\ T_0^M = S_{M1}/S_{MS}$$

, where $S_{DS}, S_{MS} =$ Design and Max spectral response accel parameter at short period

$S_{D1}, S_{M1} =$ Design and Max spectral response accel parameter at 1-sec period

$T_0^D, T_0^M =$ Design and Max spectral response period

TBC2006 specifies three design base shears based on variety of design earthquakes, which are Minimum Design Earthquake, Lower Level Earthquake and Maximum Considered Earthquake. The governing design base shear has to be the largest value of the three design base shears V (Minimum Design Base Shear), V^* (Intermediate Design Base Shear) and V_M (Maximum Considered Base Shear).

$$V = \frac{I}{1.4\alpha_y}\left[\frac{S_{aD}}{F_u}\right]_m \times W \text{ , under MDE} \qquad V^* = \frac{F_u I}{4.2\alpha_y}\left[\frac{S_{aD}}{F_u}\right]_m \times W \text{ , under LLE}$$

$$V_M = \frac{I}{1.4\alpha_y}\left[\frac{S_{aM}}{F_{uM}}\right]_m \times W \text{ , under MCE.}$$

The Design Base Shear is determined as $V = \max\{V, V^*, V_M\}$.

2.2 Structural Analysis

2.2.1 Linear Analysis

CSI-EATBS was used for structural static and dynamic analyses. The structural dynamic analysis was based on the response spectrum analysis method with adjusted lateral force according to TBC2006. Additionally, the analysis included 5% eccentricity to account for the accidental torsion. Then it considered vertical acceleration when performing dynamic analysis. The design wind load for x direction was 1315.3 tons inward, 1282.0 tons outward and 6845.7 tons-m torsion. The design wind load for y direction was 2341.1 tons inward, 901.4 tons outward and 6655.0 tons-m torsion.

The maximum story drift at x direction due to seismic force was found at 19th floor as 1.717% of story height, while the maximum story drift at y direction was found at 18th floor as 1.646% of story height. The maximum story drift at x direction due to wind force was found at 27th floor as 2.915% of story height, while the maximum story drift at y direction was found at 27th floor as 3.296% of story height. See Figure 2 for more details.

2.2.2 Nonlinear Analysis

Structural nonlinear analyses were conducted considering six pairs of time-history ground motions. Three pairs of time-story are site-specific ground motions from Kaoshiung, while the remaining included Elcentro, Kobe and Chichi earthquakes. Nonlinear analyses were performed using PISA3D (K.C. Tsai, 2008), although the vertical acceleration was not considered.

2.2.3 Comparison of Linear and Nonlinear Analyses

According to response spectrum analysis, three fundamental periods on x, y and vertical directions are 3.75 seconds, 3.60 seconds, and 3.20 seconds, respectively. An amplification factor for torsions was calculated and included for final analytical results. To meet the design criteria, the Demand/Capacity ratio was controlled within 0.95. The differences on mode shapes using ETABS and PISA3D are very small, based on Table 1 below.

Table 1 Comparison of ETABS and PISA3D on Structural Period

	ETABS	PISA3D
1st mode	3.75 s	3.74 s
2nd mode	3.60 s	3.59 s
3rd mode	3.20 s	3.21 s
4th mode	1.21 s	1.21 s
5th mode	1.18 s	1.18 s
6th mode	1.11 s	1.08 s

2.3 Structural Components for Retrofit

2.3.1 Member Dimension

The typical thickness of the floor slab is 15 cm while some are 20 cm. Typical steel beam members are H693x14.5x23.6 ~ H700x450x16x38. Typical steel column members are Box 900x900x32 ~ Box 900x900x50, Box 800x800x40 ~ Box800x800x45 and Box700x700x35. Typical columns from the ground floor to the 9th floor are composite members covered with concrete up to 1300x1300 mm.

2.3.2 Structural Materials

The typical reinforced concrete compression strength is f'_c=280 kg/cm^2 ,while the typical composite concrete strength is f'_c=210 kg/cm^2. The new steel beams, columns and braces are A572 Gr 50 while the existing steel beams are A36, and columns are A572 Gr. 50. Where the P_y for BRB members is smaller than 200 tons the steel strength is A36, otherwise it is A572 Gr. 50.

3. Reinforced Beam-Column Connections

3.1 Methodology for Retrofits

Because most of the existing slabs were in placed, it was a big challenge to reinforce the beam-column connections and to increase the story height. The beam-column connections were designed to have a plastic moment capacity of M_d, where

$$M_d=1.1Z_bF_{yb}(L_b/L_s) \tag{1}$$

where Z_b is the plastic section modules, F_{yb} is the beam yielding stress and Ls is the beam span between center lines and Lb is the beam span length between faces of columns. The proposed reinforced beam-column connections then are expected to have a plastic moment capacity of M_u, where

$$M_u=Z_bF_{yb}+ Z_{PL}F_{PL} >= \alpha\ M_d \tag{2}$$

where Z_{PL} is the reinforced plastic section modules, F_{PL} is the reinforced beam yielding stress and α is assumed conservatively at 1.46 based on this study. In addition, in order to meet the strong-column-weak-beam design requirement, the ratio is within 1.13 ~ 2.58 which met the design criteria.

3.2 Test Results

Three sets of existing beam-column connections were taken directly from the construction site for ultra sound testing in the lab. The test results are listed in Table 2. Sample 1 and 3 were from the 33rd floor, which have some unacceptable erosion conditions, while sample 2 was satisfactory. Sample 3 was reinforced and strengthened to meet AISC 2002 seismic provisions. This test was conducted in Chiao-Tung University after the samples were directly removed from the existing construction site. Such procedures demonstrated the unbiased results to ensure the satisfactory design and construction quality.

Table 2 Test results of three sets of beam-column connections

Specimen Number	1	2	3
Column	32th-33th F at line 5/D	32th-33th F at line 4/D	32th-33th F at line 4/C
	Box 700x700x35x35	Box 700x700x35x35	Box 700x696x35x35
Beam	33th F at line 5/D,E	33th F at line 4/D,E	33th F at line 5/C,D
	A33G60	A33G60	A33G54
	H702x254x15.5x27.9 (W27x146)	H702x254x15.5x27.9 (W27x146)	H688x255x13.1x21.1 (W27x102)
UT (Upper beam-column joint)	Satisfactory	Satisfactory	No. Good. Unacceptable erosion level.
UT (Lower beam-column joint)	No. Good. Unacceptable level of connection.	Satisfactory	Satisfactory

3.2.1 Sample 2and 3 Test Results

Figure 3.a illustrates the satisfactory hysteretic behavior of the sample 2 beam-column connection. It indicates that the connection was able to endure up to a 0.01 radian angle before it started to yield. The maximum beam-column moment capacity was 2511 kN-m when the angle was at 0.03 radian.

Figure 3.b illustrates the unfavorable hysteretic behavior of the sample 3 beam-column connection. It indicates that the connection experienced serious plastic moment hinges when the draft angle was at 0.047 radian although the wielding remained intact. The maximum beam-column moment capacity was 1865 kN-m when the angle was at -0.03 radian.

4. BUCKLING RESTRAINED BRACES (BRB) NONLINEAR SAMPLE TESTS

4.1 Test Procedures

In order to ensure the satisfactory hysteretic behaviors and their energy dissipation capacity, three actual sizes of BRB members were tested at the Earthquake Engineering Research Center at Taiwan University. The nonlinear component test was conducted using AISC/SEAOC 2005 proposed time-history analysis procedures.

4.2 Test Results

Figures 4 and 5 demonstrate satisfactory nonlinear behaviors of the BRB members. It proves that BRB members dissipate acceptable energy under time-history ground motions. In addition, the results show that BRB also meet the designed stress-strain requirements. Overall, the proposed BRB demonstrate excellent energy dissipation capacity and function as intended damping elements.

5. STRUCTURAL NONLINEAR MODEL

PISA3D was used to analyze the three dimensional nonlinear structural models. To most utilize the efficiency of the program, the following assumptions were made: (1) the underground basement was not considered in the models; (2) smaller beam members were not included and their weight was added to the larger beams; (3) Cantilever beams were ignored, and (4) walls were not included in the models. The structural models consider three various structural systems including conventional moment frame, EBF and the BRB system.

6. EARTHQUAKE RESISTING CAPACITY

Three levels of earthquakes were identified for structural design in the 2005 Taiwan Building Code. Design Earthquake (DE) level has 475 return periods while Maximum Considered Earthquake (MCE) has 1000 year return periods. An earthquake ground motion was recorded in Han-Chung, a southern town in Taiwan, on December 26, 2006**.** Three site-specific ground motions KAUEW (EQ1), KAUPEW (EQ2) and KAUPNS (EQ3) based on that earthquake recorded by seismic monitor stations and additional Chichi (EQ4), Elcentro (EQ5) and Kobe (EQ6) ground motions were considered for time-history analyses.

6.1 Design Earthquake Level

The results from these six time-history ground motions are summarized as follows:

1. The maximum story drift of 106.4 cm was dictated on the roof floor under EQ 6 ground motion.
2. The maximum story rotation of 0.0111 radian was found at the 7th floor under EQ 6 ground motion.
3. The maximum story shear of 5672.9 tons on x direction and 6585.6 tons on y direction were found.
4. Most of plastic hinges were concentrated on the BRB elements. The maximum beam plastic rotation can reach up to 0.00983 radian, while columns do not form plastic rotations. The maximum shear plastic rotation was 0.0307 radian. The maximum of strain ε_c of BRB is about 1.46 %.

6.2 Maximum Considered Earthquake Level

The results from these six time-history ground motions are summarized as follows:

1. The maximum story drift of 118.6 cm was dictated on the roof floor under EQ 6 ground motion.
2. The maximum story rotation of 0.0131 radian was found at the 16th floor under EQ 6 ground motion.
3. The maximum story shear of 6648.0 tons on x direction and 7047 tons on y direction were found.
4. Most of plastic hinges were concentrated on the BRB elements. The maximum beam plastic rotation can reach up to 0.0129 radian, while columns do not form plastic rotations. The maximum shear plastic

rotation was 0.0408 radian. The maximum of strain ε_c of BRB is about 1.85 %.

6.3 Incremental Dynamic Analysis

An Incremental Dynamic Analysis (Vamvatsikos and Cornell, 2006) was performed to obtain the capacity curves shown in Figure 6. When the roof drift ratio was 0.25%, the structure started to yield. The maximum base shear was 9600 tons or 0.21 W and is about six times of the design base shear. Under the MCE level, the maximum base shear was 7000 tons, which is about 72% of the maximum base shear using IDA. The result indicated that the structure has enough ductility for MCE earthquakes.

7. CONCLUSIONS

The following conclusions are summarized:

(1) The proposed retrofit project was made possible after the stringent design process, test verification, linear structural analysis, dynamic analysis, and capacity curve analysis were conducted to ensure satisfactory enhanced structural seismic behaviors.
(2) BRB and EBF provide excellent stiffness needed to resist lower earthquakes and wind load. BRB and EBF provide satisfactory energy dissipation for DE and MCE earthquakes. Particularly, BRB has been proven to be an alternative element for seismic resistance.
(3) Reinforced beam-column connections were tested to show their superior plastic moment hinge capacity.
(4) The various sample test results help to identify which beam-column connections are acceptable.
(5) The retrofit strategy was proved to be innovative and made the project possible from both an engineering and financial perspective.
(6) Using static analysis, response spectrum analysis, time-history analysis, and incremental dynamic analysis and nonlinear analysis, the results lead to verification of the satisfactory behaviors of BRB and ERF members during earthquakes.

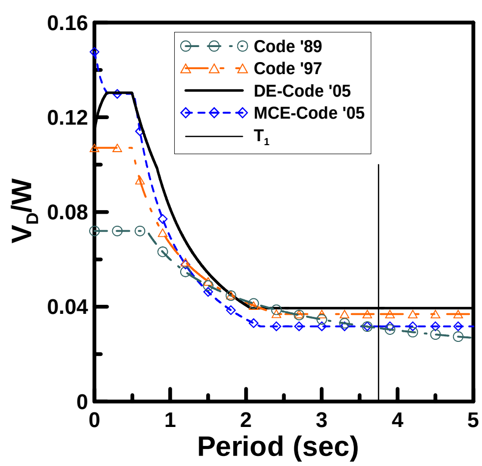
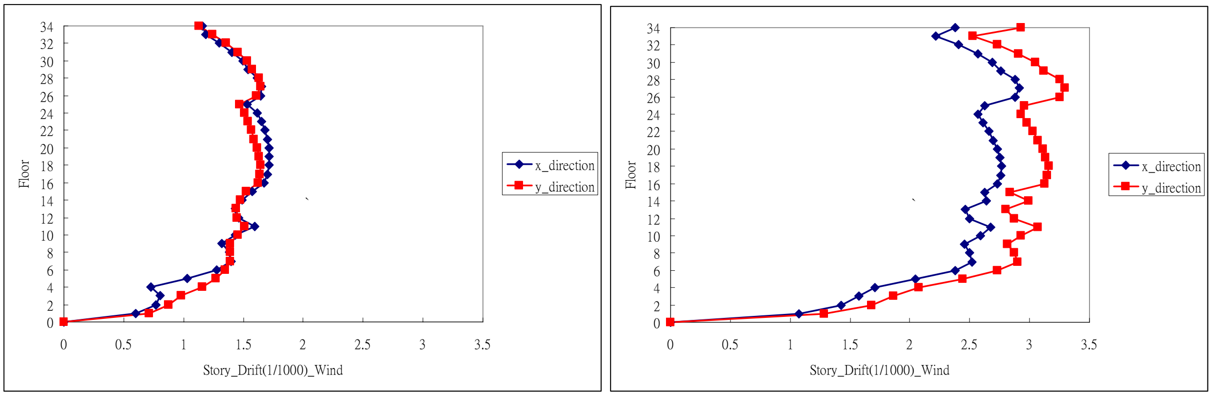


Figure 1. TBC seismic design force



Seismic design force
X-direction (max)=1.717/1000 (19th F)
Y-direction (max)=1.646/1000 (18th F)

Wind force
X-direction (max)=2.915/1000 (27th F)
Y-direction (max)=3.296/1000 (27th F)

Figure 2. story displacements under seismic design force and wind force

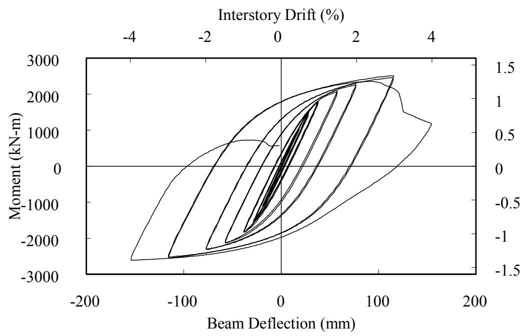


Figure 3a. Sample 2 hysteretic curve

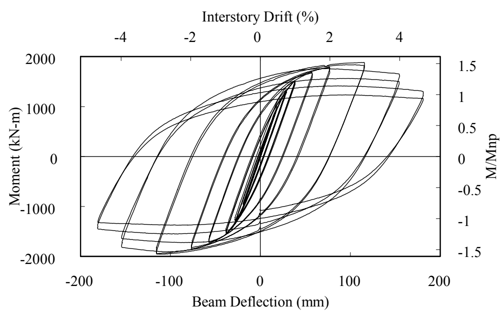


Figure 3b. Sample 2 hysteretic curve

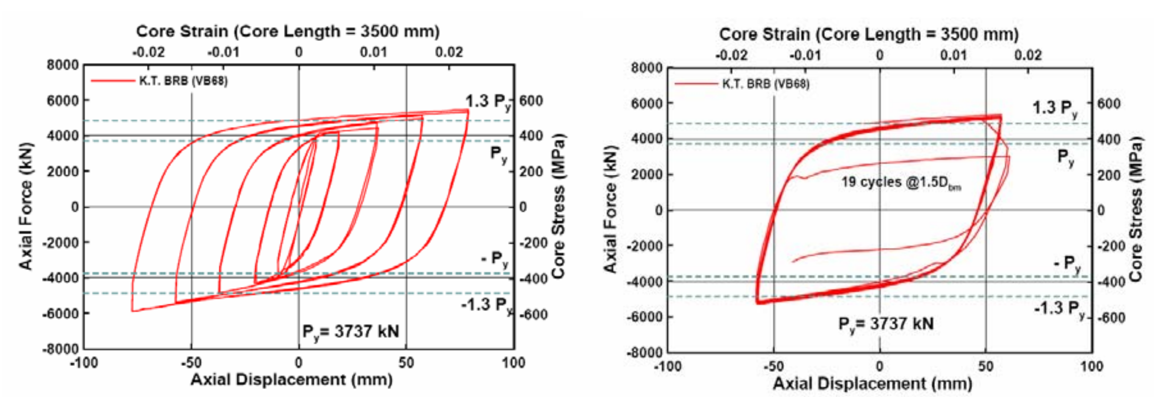


Figure 4. VB68 Axial force and axial displacement curve

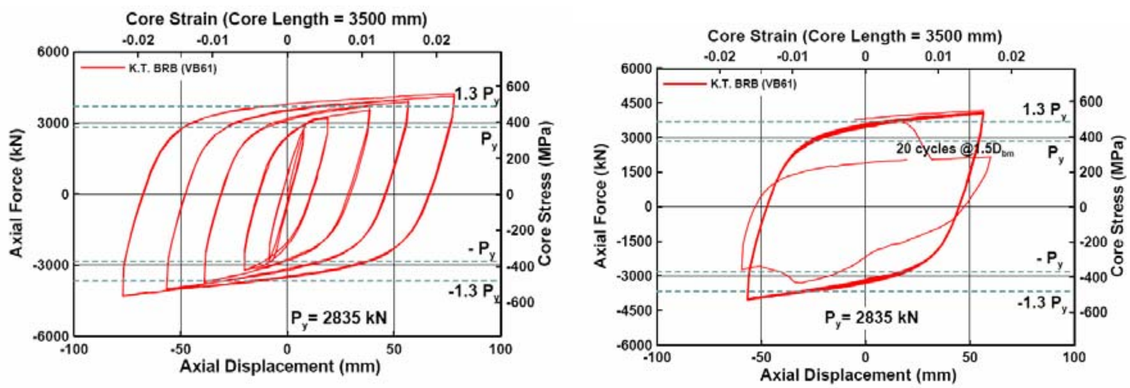


Figure 5. VB61 Axial force and axial displacement curve

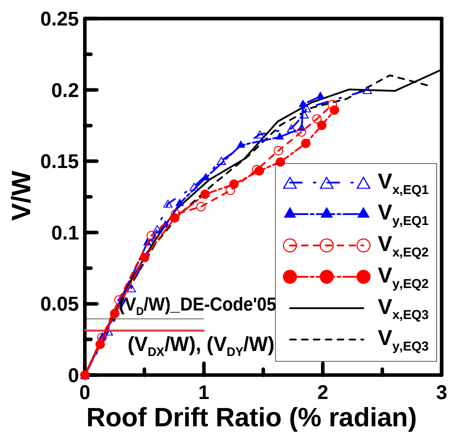


Figure 6. Incremental Dynamic Analysis

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