PSEUDO DYNAMIC EXPERIMENTAL RESPONSES OF A FULL SCALE CFT/BRB COMPOSITE FRAME

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Abstract

A full scale 3-story 3-bay concrete filled steel tube (CFT) column and buckling restrained braced (BRB) composite frame was tested using pseudo dynamic testing procedures and four earthquake accelerations. The key features of the structural system include using three different types of beam-to-CFT column moment connection and three different types of BRB. The CFT-BRB frame was designed using the displacement based seismic design procedure considering a target inter-story drift limit of 0.02 and 0.025 radians for the 10% and 2% chances of exceedance in 50 years, respectively. The pseudo dynamic responses of the specimen observed during the application of the earthquake load effects are discussed. During the first test, the gusset for the 1st story brace to beam connection was buckled. The buckled gusset was then heat straightened before welding stiffeners at the free edges of the gusset at the brace to two column bases and at all the brace to beam connections. The pseudo dynamic tests were successfully completed with the experimental responses satisfactorily predicted by analysis conducted before the tests. This experimental program illustrates that the experimental response data and the video images of the specimen has been effectively disseminated through the Internet during and after the tests.

INTRODUCTION

Through international collaboration between researchers in Taiwan, Japan, and the United States, a full-scale 3-story 3-bay RC column and steel beam RCS composite moment frame was tested in October of 2002 in the structural laboratory of National Center for Research on Earthquake Engineering (NCREE) in October 2002 (Chen et al. 2003). In the year 2003, a full-scale 3-story 3-bay CFT column with the buckling restrained braced composite frame (CFT/BRBF) specimen has been tested in October in a similar manner. The 3-story prototype structure is designed for a highly seismic location either in Taiwan or United States. The typical bay width is 7m and typical story height is 4m. The total height of the frame, including the footing, is about 13m. The 2150mm wide concrete slab is adopted to develop the composite action of the beams. Measuring 12 meters tall and 21 meters long, the specimen is among the largest frame tests of its type ever conducted. The frame has been tested using the pseudo-dynamic test procedures applying input ground motions obtained from the 1999 Chi-Chi and 1989 Loma Prieta earthquakes, scaled to represent 50%, 10% (DBE), and 2% (MCE) in 50 years seismic hazard levels. Following the pseudo-dynamic tests, since none of the brace was fractured, quasi-static loads have been applied to cyclically push the frame to large inter-story drifts up

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to the failure of the braces. It provides valuable data to validate possible failure mechanism and analytical models for large deformation response. Being the largest and most realistic composite CFT/BRB frame ever tested in a laboratory, the test provides a unique data set to verify both computer simulation models and seismic performance of CFT/BRB frames. This experiment also provides great opportunities to explore international collaboration and data archiving envisioned for the Networked Earthquake Engineering Simulation (NEES) or the Internet-based Simulations for Earthquake Engineering (ISEE) (Wang et al. 2003) research programs launched recently in USA and Taiwan, respectively. This paper summarizes the displacement-based seismic design procedures adopted in the design of CFT/BRB frame specimen. During the planning stage, extensive nonlinear dynamic analyses were also carried out in order to ensure the possible seismic demands would not exceed the force and displacement limits of the test facility. The analytical predictions were broadcasted along with the real time experimental results during each test. This paper describes the experimental and analytical results and evaluates the seismic performance of the frame specimen.

DISPLACEMENT BASED SEISMIC DESIGN OF THE CFT/BRB FRAME

The 3-story CFT/BRB frame shown in Fig. 1 is employed in this experimental research. The prototype three-story building consists of 6-bay by 4-bay in plane. In the two identical prototype CFT/BRB frames, only the two exterior beam-to-column joints (Fig. 1) in each floor are moment connections, all other beam-to-column connections are assumed not to transfer any bending moment. The details of the moment connections are schematically given in Fig. 2 for the top through the first floor beams. The BRBs are installed in the center bay. Square CFT columns are chosen for the two exterior columns while the center two columns are circular CFTs. Story seismic mass is 31.83 ton for the 1st and 2nd floors, 25.03 ton for the 3rd floor for each CFT/BRB frame (half of the building). All steel is A572 GR50 with an yield strength of 350 Mpa and the infill concrete in the CFT columns are 35 MPa. The multi-mode displacement-based seismic design (DSD) procedures adopted for this frame specimen have been studied and can be found in the reference (Weng 2003). But it is found that the results are not very different from applying the DSD procedures proposed by others (Loeding et al 1998; Medhekar and Kennedy 2000) assuming that the CFT/BRB frame specimen vibrates essentially in a single mode. The detailed DSD details of the specimen can be found in the reference (Tsai et al. 2003). It includes:

1. Select acceptable (target) maximum story drift levels. Figures 3a and 3b consider Taiwan seismic code draft updated in 2002. The 5% damped S_a values for TCU082EW (1999 Chi-Chi Taiwan Earthquake) record are also shown on Figs. 3a and 3b. The corresponding PGA values for the 10/50 (DBE) and 2/50 (MCE) levels of excitations are 0.46g and 0.62g, respectively, for the TCU082EW record. Similarly, for the LP89g04NS (1989 Loma Prieta) record, the corresponding PGA values for the 10/50 and 2/50 levels of excitations are 0.42g and 0.54g, respectively. For the 10/50 and 2/50 events, the inter-story drift limits are set at 0.02 and 0.025 radians, respectively.
2. Calculate the maximum displacement profile. It is assumed that structural first modal design displacement profile can be simplified as an inverted triangle. Under the 10/50 and 2/50 events, the story drift limit is 0.02 and 0.025 radians, respectively. The corresponding target roof displacements are 24cm and 30cm for the 10/50 and 2/50 events, respectively.
3. Calculate the system displacement for the two events. This step essentially translates the actual MDOF structure to the substituted SDOF structure through displacements.
4. Estimate system ductility from the properties of buckling restrained braces. The relationship

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between the inter-story drift and the deformation of the BRB is approximated as given in Fig. 4. The inelastic axial strain demand in the BRB can then be estimated considering the ratio of its energy dissipating segment length and the work point to work point dimension. The profile of the BRB is given in Fig. 5. The ductility demand on the BRB in each story can be estimated given the target inter-story displacement computed in Step 3. The averaged (of the three stories) system ductility demands imposed by the DBE and MCE events are 7.28 and 9.10, respectively.

5. Compute the effective structural vibration period. This is done first by applying the R_y-m-T relationships suggested by Newmark and Hall (1982) on the elastic design acceleration response spectra given in Figs. 3a and 3b to construct the inelastic displacement response spectra (IDRS) given in Fig. 6 for the two ductilities computed in Step 4. The effective periods under the DBE and MCE excitations can be found by intersecting the effective target roof displacements (Step 3) with the IDRS as shown in Fig. 6.
6. Calculate the effective mass.
7. Calculate the effective stiffness K_{eff}. Effective stiffness of the frame under the two events is computed using the effective mass and periods obtained from Steps 5 and 6.
8. Calculate the design ultimate and yield base shears. Two ultimate base shears are computed by multiply the two effective stiffnesses (Step 7) and the corresponding system displacement obtained in Step 3. Design yield shears are calculated from the ultimate base shears and the corresponding system ductility assuming the system post-yield stiffness is 10% of the initial elastic stiffness. It is found that the design yield base shear is larger for MCE than that for DBE, therefore MCE governs the design.
9. Distribute the design base shear over the frame height. The design yield base shear is distributed following the triangular deformed structural shape assumed in Step 2.
10. Conduct structural analysis and design the members for the CFT/BRB frame. Assuming the BRBs in each floor resist about 80% of the story shear, the cross sectional area of the brace core is computed for all BRBs. After a few iterations, it is found in the elastic model that the final selection of structural members (Table 1) considering the actual material coupon strength (Table 2) satisfies all the requirements noted above. In particular, the BRBs in each story will reach yielding at the proximity of the design story shear. The total double-cored cross sectional area for each individual set of brace is 15, 25 and 30 cm^2 for the 3rd, 2nd and 1st story, respectively. The supporting beams above the BRBs satisfy the capacity design principal considering the strained hardened BRBs and an unbalanced vertical load resulted from the difference of the peak BRB compressive and tensile strengths. The fundamental vibration period of the four design results noted above ranges from 0.68 to 0.72 second. In addition, the effects of varying the flexural stiffness of the CFT columns from fully composite (using LRFD specification) to the bare steel have been found insignificant, only change the fundamental vibration period from 0.68 to 0.71 second.

Three different types of moment connections, namely through beam, external diaphragm and bolted end plate types, varying from the first floor to the third floor were fabricated for the exterior beam-to-column connections (Fig. 2). Three types of BRBs, including the single core, double-cored and the all-metal BRBs, were adopted in the three different stories. In particular, two single-cored unbonded braces (UBs), each consisting of a steel flat plate in the core, were donated by Nippon Steel Company and installed in the second story. Each UB end to gusset connection uses 8 splice plates and 16-24mmϕ F10T bolts. The two BRBs installed in the third story are double-cored constructed using

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cement motar infilled in two rectangular tubes (Tsai et al. 2003) while the BRBs in the first storey are also double-cored but fabricated with all-metal detachable features (Tsai and Lin 2003). Each end of the double-cored BRB is connected to a gusset plate using 6- and 10-22mmϕ F10T bolts at the third and first stories, respectively. No stiffeners were welded to the free edges of any gusset before the testing.

EXPERIMENTAL PREPARATION

Inelastic static and dynamic time history analyses have been conducted using PISA3D (Tsai and Lin 2003) and OpenSees (Open System for Earthquake Engineering Simulation), developed at National Taiwan University and Pacific Earthquake Engineering Research Center (PEER), respectively. In the PISA3D model, all BRBs were modeled using the two-surface plastic (isotropic and kinematic) strain hardening truss element. All the beam members were modeled using the bi-linear beam-column elements. The CFT columns were modeled using the three-parameter (stiffness, strength and pinching degradation) degrading beam-column element. The P-M curve of the CFT columns was constructed using EC4 (CEN 1992) before multi-linearized. In the OpenSees model, all the CFT columns and steel beams of the frame were represented using the flexibility-based nonlinear beam-column element with discretized fiber section model. All the BRBs were modeled using the truss element with bilinear isotropic strain hardening. A leaning column has been introduced in both the PISA3D and the OpenSees models in order to simulate the 2nd order effects developed in the gravity columns. Two ground acceleration records, 1999 Chi-Chi Taiwan and 1989 Loma Prieta Earthquakes as shown in Fig. 7 scaled to various hazard levels were utilized in the nonlinear time history analysis. Four earthquake load effects of three different hazard levels, namely 50/50 (using TCU), 10/50 (using LP), 2/50 (using TCU), another 10/50 (using LP) were scheduled for the frame test (http://cft-brbf.ncree.gov.tw). Predicted floor displacement and story shear time histories were webcasted before, during and after the test. Analytical results suggest that the arrangement of 4, 3 and 3 actuators, each having ±980kN force and ±50cm stroke capacities, is appropriate for the 1st, 2nd and 3rd floor, respectively. Each footing was anchored down using four 69mm diameter post-tension bars to the strong floor. A displacement reference frame was erected as shown in Photo 1 in order to accurately impose the target displacements onto to the specimen.

NETWORKED PSEUDO DYNAMIC TESTING TECHNIQUES

During the pseudo dynamic tests, several computers were networked to do various tasks: including computing the target displacements (analysis engine), drive the servo-controller (facility control), perform the data acquisitions, distributing experimental and video data to Internet viewers. This is achieved from a Platform for Networked Structural Experimental (PNSE) developed for the Internet-based Simulation for Earthquake Engineering (ISEE) launched in NCREE (Wang et al. 2003). The Newmark explicit numerical integration scheme with a time step size of 0.005 second was adopted. During the pseudo dynamic tests, each time step 0.005 second cycle took about 1.0 second to execute but required about 4.0 second to write the 460 channels of experimental data to the hard disk in the data acquisition computer. In order to save time, complete data recording was executed at every 4 cycles of 0.005 second time step. In addition, while writing the data to the hard disk, the actuators were also imposing next time step's target displacements. Therefore, for an earthquake duration of 40 seconds, it took 8000 integration steps but only 2000 steps of experimental data were completely acquired and some of the key data were made available to the Internet viewers. Without counting the stopping to inspect the specimen, it took about (1+1+1+4) seconds $\times$ 2000 steps = 14,000 seconds (about 4 hours) to finish a pseudo dynamic of the CFT-BRB frame under a 40-second

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earthquake load effects. In each test, the leaning columns' second order effects were incorporated into the computation of the target displacements by modifying the equation of motion as follows:

$$m\ddot{x} + c\dot{x} + \hat{R} = -ma_g \tag{1}$$

and

$$\hat{R} = R - V_i X_i / H_i \tag{2}$$

where R is the restoring force matrix constructed from the actuator load cell readings, V_i, X_i, and H_i are the gravity load, lateral displacement and the height of the i^{th} floor respectively.

EXPERIMENTAL AND ANALYTICAL RESPOSNES

As noted above, four earthquake ground accelerations scaled to three different PGAs were planned for the PDT of the CFT/BRB frame specimen. However, in the Test No. 1, due to the buckling of the gusset plate occurred at the brace to beam connection in the first story, stiffeners were added at the free edges of all the gusset plates underneath the floor beams. A total of six PDTs were conducted before the final cyclic loading test.

Test No. 1 (50/50, TCU082EW, PGA=0.276, Oct. 3)

The PDT was stopped at the time step of 9.25 seconds without finding anything unexpected. Some bangs resulted from the slippage of the bolted beam connections were experienced. However, at the time step of 12.3 second, the out of plane buckling of gusset plate (Photo 2) at the north BRB-to-beam connection was detected at the first story. It was decided to install stiffeners at the free edges of the three gusset plates underneath the floor beams as shown in Fig. 8 before further tests. The north BRB in the first story was removed from the frame and the buckled end was heat straightened. After the buckled gusset at the first story was straightened, five stiffeners each of 12 mm thick and 200 mm wide were welded to the gusset's free edges and the web of the supporting beam (Fig. 8) at all floors. The straightened top end of the first story north BRB was reversed and connected to the column to foundation joint. The repair of the gusset, the installation of the stiffeners and the reinstallation of the north BRB at the first story were completed before October 5. The analytical predictions and the experimental responses broadcasted through the Internet are given in Fig. 9.

Test No. 2 (50/50, TCU082EW, PGA=0.276g, Oct. 5)

Test resumed using the same ground accelerations as that for Test No. 1 on October 5, the analysis predicted the experimental roof displacements satisfactorily as shown in Fig. 10. The experimental peak story drift reached 0.0052, 0.0043, 0.0038 radians respectively. Some minor flaking of the white wash near the beam-to-column moment connection was observed. All the six beam-to-column moment connections appeared to remain essentially elastic. More but still scattered bangs resulted from the bolt slippages were evident during the test.

Test No. 3 (10/50, LP89g04NS, PGA=0.426g, Oct. 5)

The subsequent PDT continued on the same day using the ground accelerations and the intensity as planned. Fig. 11 indicates that the analysis predicted the specimen's roof responses rather satisfactorily. During the tests, the concrete slab near the two brace-to-column joints was crushed. More frequent bangs resulted from the bolt slippage were evident during the test. Some additional but minor flaking of white wash near the second floor north beam-to-column moment connection was observed.

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Test No. 4 (2/50, TCU082EW, PGA=0.622g, Oct. 6)

Using the strongest earthquake load effects in this series of the PDT, test No. 4 was started and continued smoothly as shown in Fig. 12 for the roof displacement time history. At the time step of about 5.0 second, the bangs started to present. The PDT was stopped at the time step of 12.54 second in order to photograph the experimental observations. Cracks on the top of concrete foundation near the gusset plate for the south BRB-to-column joint were observed. Moreover, the slightly-bent situation of the north BRB was observed near the north BRB-to-column joint where heat straightening had applied after the Test No.1. It was decided to stop this test and stiffeners be added at the free edges of the gussets at the two brace-to-column base connections (Photo 3) before further testing. One pair of angles (Photo 4) was installed bracing the stiffener to the two anchoring steel blocks in order to prevent further out of plane deformation of the slightly-bent section of the north BRB.

Test No. 5 (2/50, TCU082EW, PGA=0.622g, Oct. 6)

The tests resumed on the Oct. 7 by applying the same earthquake accelerations as that for Test No. 4. More frequent bangs resulted from the bolt slippage were evident during the test. At the time step of about 27.0 second, a loud but relatively low sound was experienced. In addition, a sudden actuator load drop was also observed. However, the test was continued with the specimen’s strength recovered (Fig. 13). The test was successfully completed and the experimental time history responses were accurately predicted by the two analytical models (Fig. 14). After the test was completed, it was found that the concrete footing supporting the first story south BRB to column base was cracked at the corner (Photo 5). It has resulted in the prestress loss in the tie downs and the subsequent slippage of the footing by 15mm toward north.

Test No. 6 (10/50, LP89g04NS, PGA=0.426g, Oct. 7)

After un-tighten the four tie downs of the footing, the footing was pull back toward south direction by 11mm using an actuator force of about 1000kN applied at the 2nd floor. A steel cross beam was placed over the cracked side of the footing before re-applying the prestress in the four tie downs. The test was resumed and completed with the experimental responses accurately predicted (Fig. 15).

Test No. 7 (Cyclic Increasing Story Drifts from 0.01 to 0.025 radian, Oct. 8)

After stiffening the gussets and the application of six earthquake load effects, the peak story drift demand reached 2.5% at the second floor during the Test 4. However, it appears that the beam-to-CFT column moment connections have undergone relative minor nonlinear deformations and all UBs and BRBs have not been damaged. High-mode buckling of south BRB (all metal construction) at the first story was visible. It was decided that cyclic increasing uniform inter-story drifts (4 cycles 0.01, 4 cycles of 0.0125, 4 cycles of 0.02, 4 cycles of 0.025 and 2 cycles of 0.0375 radians) be imposed on the specimen to examine the failure modes. At the 4th 0.02 radian cycle, the gusset buckled out-of-plane at the brace-to-column joint for 3BRBS (the south BRB in the 3rd story). The 1BRBS high-mode had also buckled severely. On the first 0.025 radian cycle, the top end of the two UBs in the 2nd story had been found fractured and deformed the supporting beam laterally at the gusset to 3rd floor beam connection. The cyclic tested was therefore stopped.

Repairing the Buckled Gussets and Replacing the BRBs for the Phase-2 Tests

The twisted gusset under the 3rd floor beam had been removed before installing a new one. In addition, stiffeners were welded at the free edges of the gusset at the brace to column joints. Six new BRBs, two all metal double cored construction for the 1st story, four concrete filled double cored for the 2nd and

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3rd stories, have been installed. Further tests have been conducted on October 31 and November 1. All the key test results are available through the web site (http://cft-brbf.ncree.gov.tw). Test results have confirmed that welding the stiffeners at the free edges of the gusset are extremely effective in preventing the buckling to the gusset.

CONCLUSIONS

Based on the test and analytical results, summary and conclusions are made as follows:

- Since most the story shear is resisted by the BRBs, test results confirm that the global dynamic responses of the 3-story 3-bay CFT-BRB frame specimen can be satisfactorily predicted using both the two analytical models presented herein. This is primarily because the nonlinear responses of the BRB can be accurately represented by the elastic strain hardened constitutive models adopted in the two types of truss elements.
- The peak story drift reached 0.025 radian after applying the 2/50 design earthquake on the specimen. It appears that the DSD procedure adopted in the design of the specimen is effective in limiting the ultimate story drift under the effects of the design earthquake.
- CFT/BRBF performed extremely well after the application of six earthquake load effects. Very minor changes on stiffness and damping are observed as evidenced from the free vibration tests conducted after each earthquake pseudo dynamic test.
- Stiffeners added along the free edges of the gusset plate are effective in preventing out-of-plane instability of the brace-to-column connections. However, it also introduces flexural demands on the BRBs. Further research is needed to study the BRB end connections.
- All the moment connections survived all the Phase-1 and Phase-2 tests without failure. The BRBs effectively control the story drift and reduce the nonlinear demand imposed on these moment connections.
- Tests confirmed that the PNSE architecture implemented for the ISEE is very effective in disseminating real time test results through the Internet.

ACKNOWLEDGEMENTS

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Table 1 Selection of member sizes and grades

Member	Beam Sizes and Core Cross Sectional Area of Braces (A572 GR50)		
Location	1FL	2FL	3FL
Beam (mm)	H400×200×8×13	H450×200×9×14	H456×201×10×17
Brace (cm^2)	30	25	15
Dimension of Columns (A572 GR50) unit : mm		CFTs: C1: Tube: 350×9, C2: Pipe: 400×400×9	

Table 2 Material strength

		Positions of Sampling		f_y (MPa)	f_u (MPa)
Steel (A572Gr.50)	3FL	Beam	Flange	372	468
			Web	426	493
		BRB3	core steel material	373	483
	2FL	Beam	Flange	414	503
			Web	482	538
		BRB2	core steel material	397	545
	1FL	Beam	Flange	370	486
			Web	354	485
		BRB1	core steel material	421	534
C1(Tube-400-9)		Steel		374, 488	488
C2(Pipe-400-9)		Steel		543	584
Concrete		f_c'=35 MPa			

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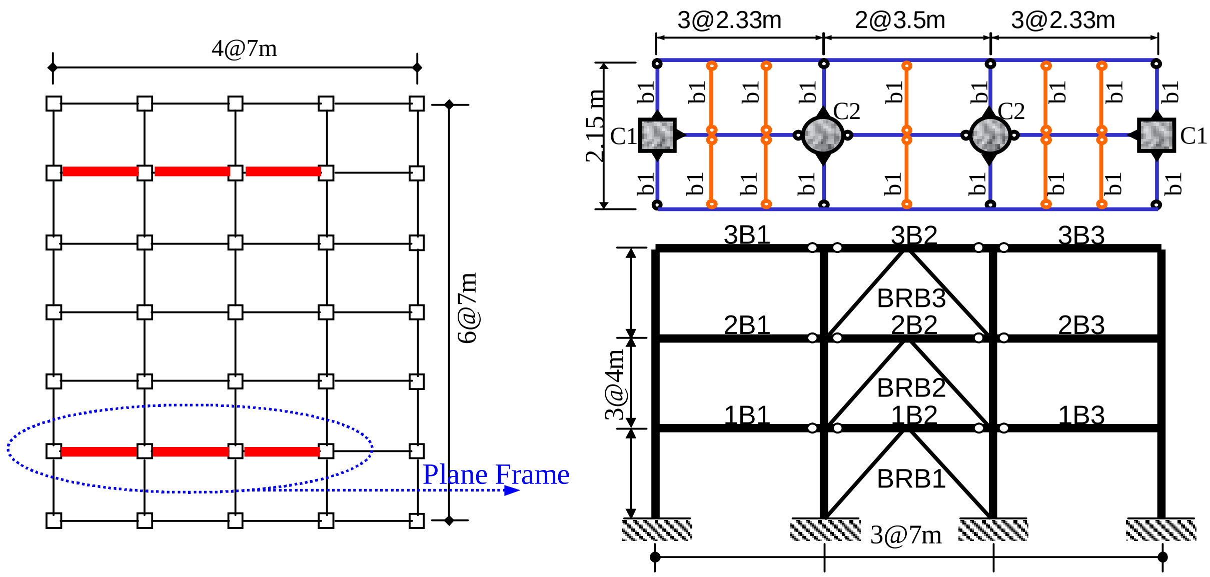

Fig.1 Floor framing plan and elevation

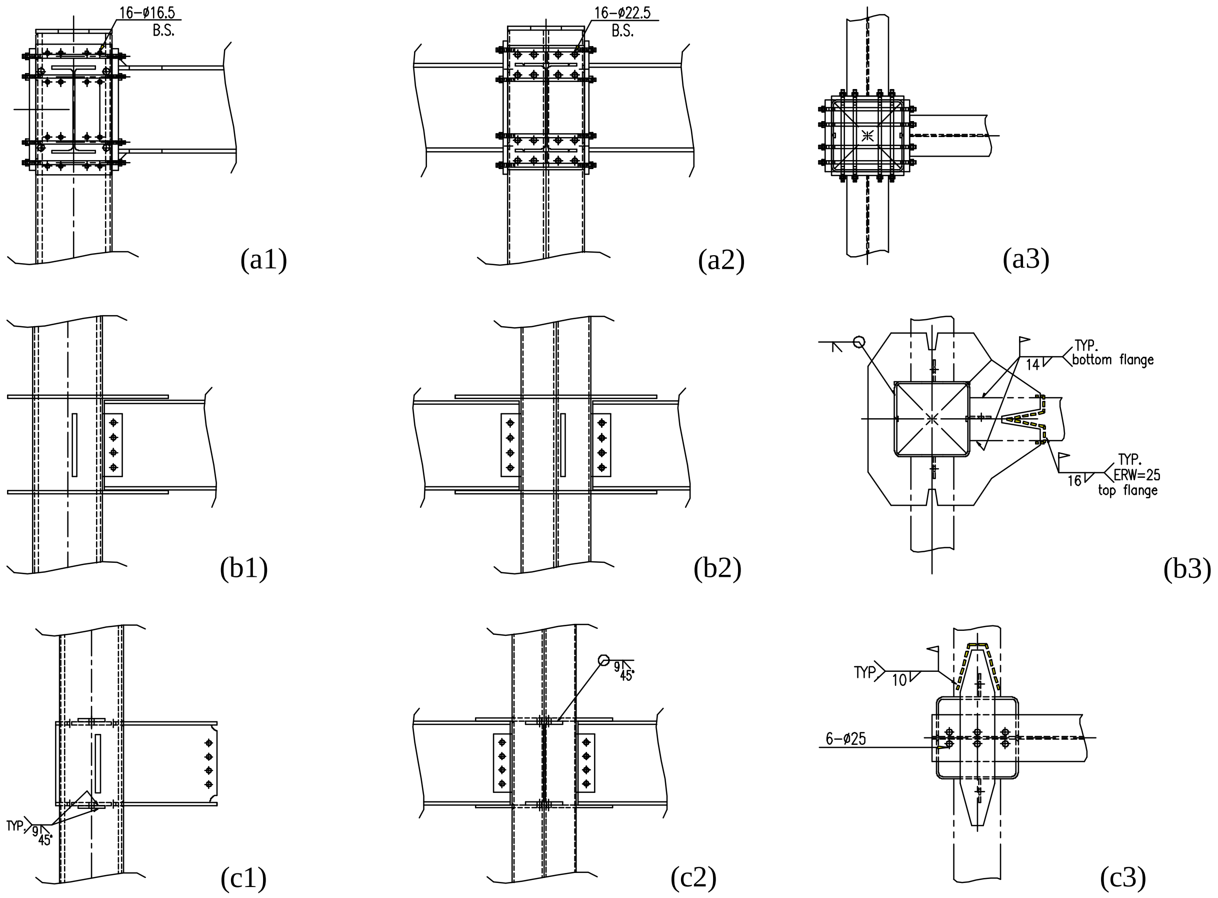

Fig.2 Moment connection details (a, b, c for 3rd, 2nd and 1st floor, respectively)

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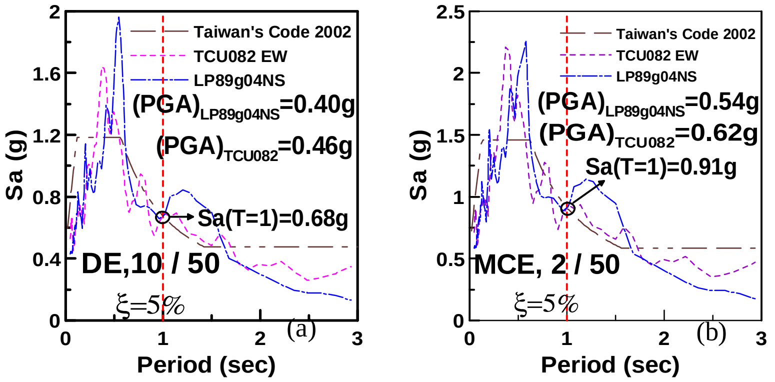

Fig. 3 Design acceleration spectra (a)10/50 (b)2/50 hazard level

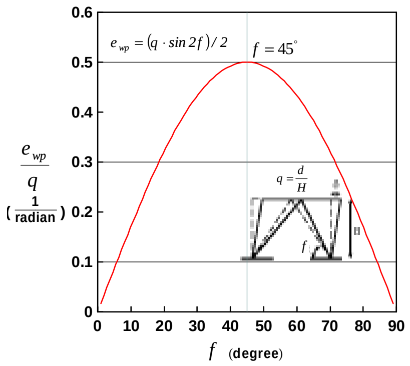

Fig. 4 brace strain versus story drift relationships

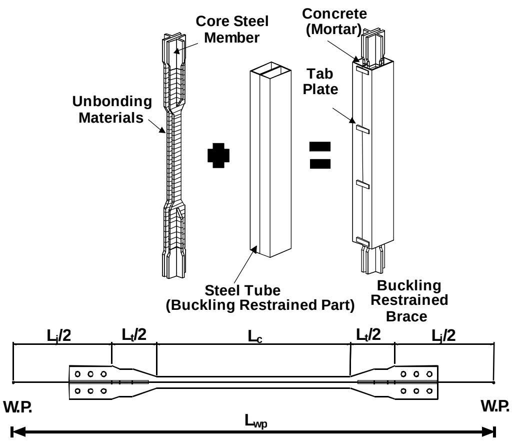

Fig. 5 Profiles of core steel in the BRB

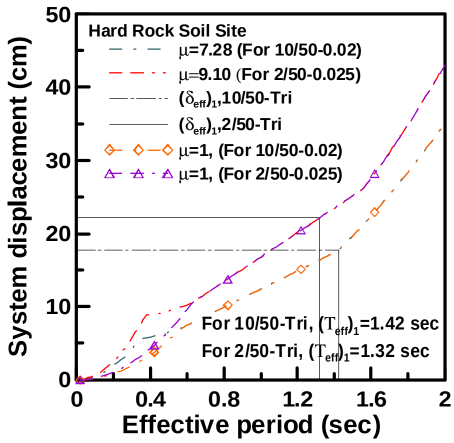

Fig.6 Inelastic Design Displacement Spectra

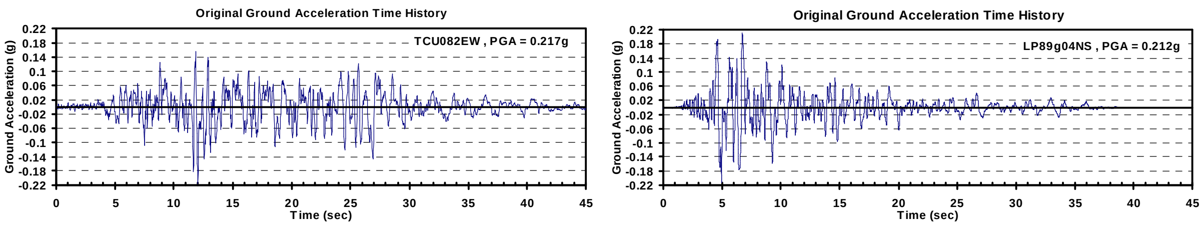

Fig. 7 Original ground accelerations used in test (before scaling)

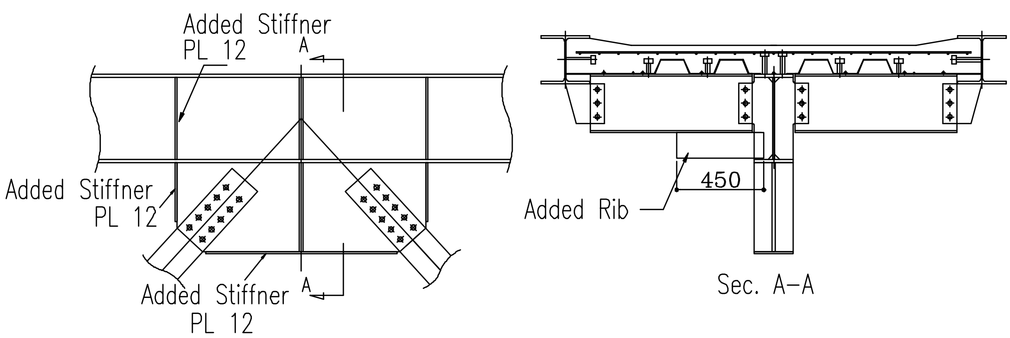

Fig. 8 Added Stiffeners and ribs after the Test No.1

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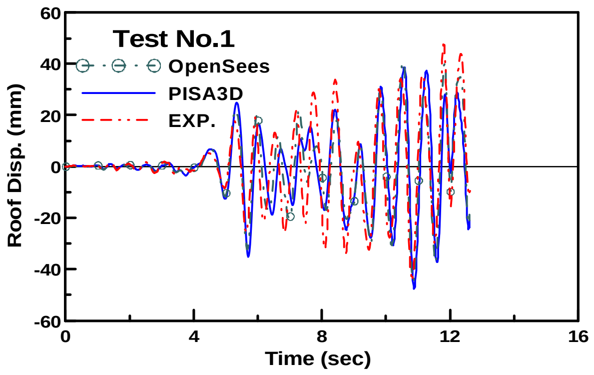


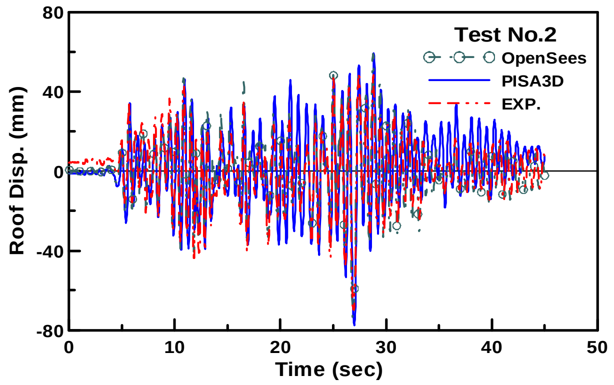
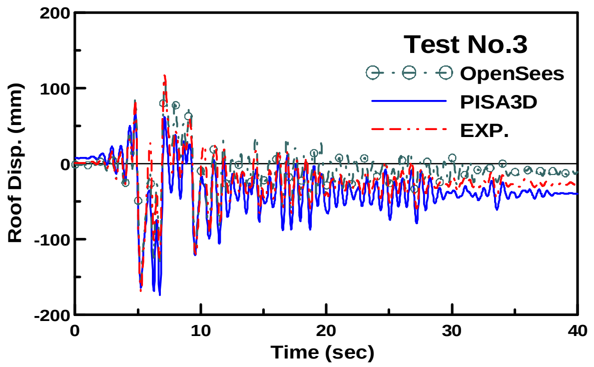


Fig. 9 Roof displacement time history in Test No.1

Fig. 10 Roof displacement time history in Test No.2



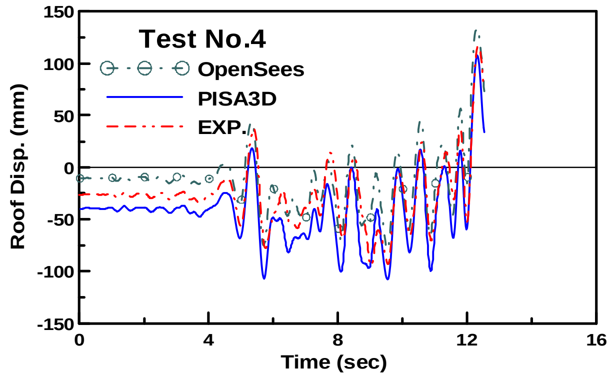
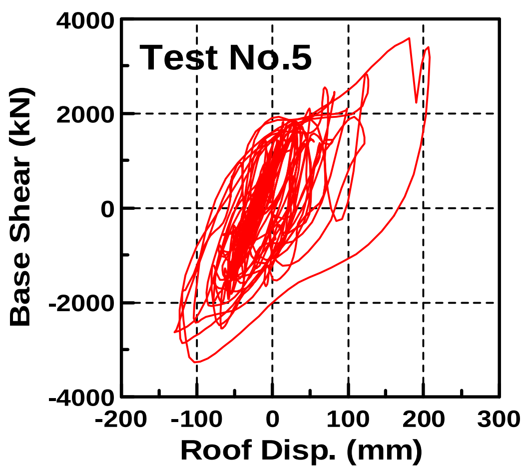


Fig. 11 Roof displacement time history in Test No.3

Fig. 12 Roof displacement time history in Test No.4



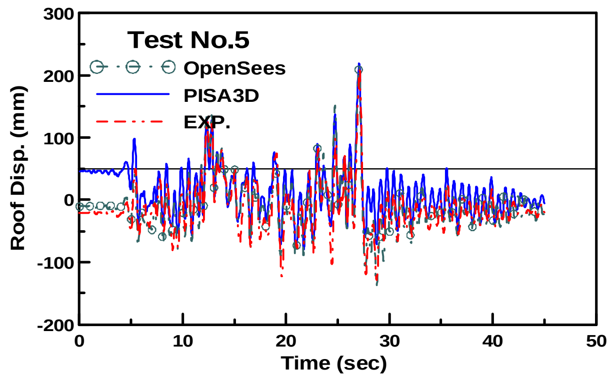


Fig. 13 Roof Displacement versus base shear in the PDT

Fig. 14 Roof displacement time history in Test No.5

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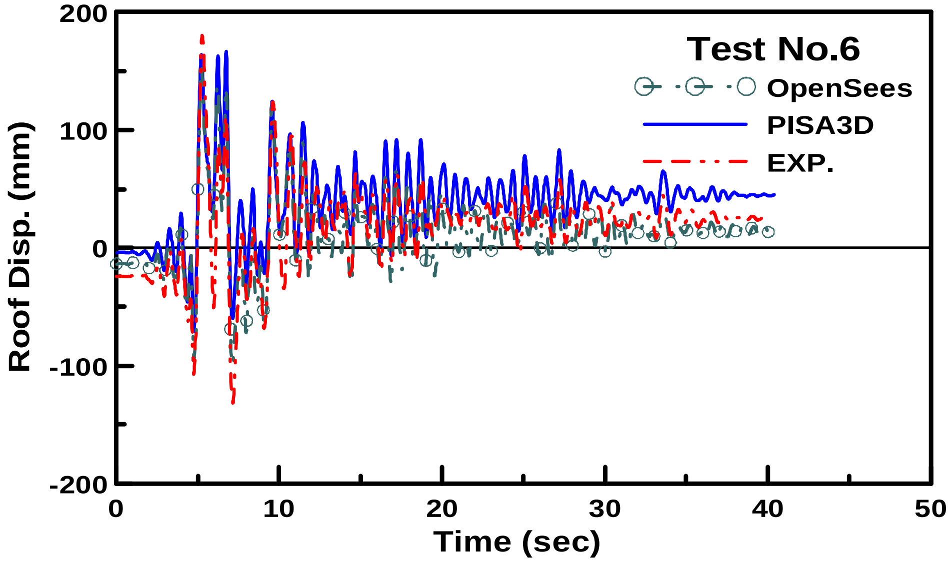

Fig. 15 Roof displacement time history in Test No.6

Photo 1 Experimental set up of the PDT of a full scale 3-story 3-bay CFT-BRB composite frame

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Photo 2 Out of plane buckling of gusset plate in the Test No.1

Photo 3 Stiffeners be added at the free edges of the gussets at the two brace-to-column base connections after the Test No.4

Photo 4 One pair of angles was installed bracing the stiffener after the Test No.4

Photo 5 The concrete footing supporting the first story south BRB to column base was cracked at the corner after the Test No.5

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