ANALYTICAL ASSESSMENT OF A 2-STORY BRBF FOR FULL-SCALE 3D SUB-STRUCTURAL PSEUDO-DYNAMIC TESTING

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ABSTRACT

In this paper, the analytical studies of a 2-story buckling-restrained-bracing frame (BRBF) to be tested using sub-structural pseudo-dynamic testing (PDT) procedures are described. The seismic ground acelerations considered in this study was recorded in the 1999 Chi-Chi earthquake. The ground accelerations are scaled up to represent 50%, 10%, and 2% probability of exceedance in 50 years and bi-directionally applied in the transverse and longitudinal directions simultaneously. The displacement-based seismic design procedures used in the design of the BRBF is presented. A systematic approach to determine the earthquake scenario for the test is developed. For the first phase of the test, analytical results suggest that the peak story drift is likely to reach 0.025 radian in the transverse direction under the 2/50 exceeding-probability earthquake. It also shows that the allocation of actuators in the BRBF direction might be adequate. For the second phase of the test, the analytical model is adjusted to form three planar asymmetric structures. These three asymmetric structures are coupled in translational and rotational motions with different extent. One of the objectives of the Phase II test is to validate the η-MPA procedures in estimating the seismic demands of asymmetric structures. The analytical results obtained by η-MPA procedures and response history analysis (RHA) are compared.

INTRODUCTION

In March 2005, sub-structural pseudo-dynamic tests of a full-scale steel buckling restrained braced frame (BRBF) with internet testing techniques will be conducted at the National Center for Research on Earthquake Engineering (NCREE) in Taiwan. The prototype of the 2-story building configuration and the BRBF specimen is shown in Fig. 1. In the context of the pseudo dynamic testing, only one BRB frame specimen will be tested, the remaining structure will be simulated analytically. The prototype structure is located at seismic zone I, which has strong seismic intensity according to Taiwan Building Regulations. The bay width and story height of the BRBF are 8m and 4m, respectively. This experimental program consists of two phases. In Phase I, the seismic ground motion records, which was recorded in the 1999 Chi-Chi earthquake, are scaled up to represent 50%, 10%, and 2% probability of exceedance in 50 years (denoted as 50/50, 10/50 and 2/50 events, respectively). The ground motion will be applied bi-directionally in the transverse (Y-axis) and longitudinal (X-axis) directions. One of the test objectives in Phase I is to observe the performance of various BRB to gusset plate connections (Fig. 2) under the bi-directional seismic load effects (Tsai et al., 2005). In Phase II, the prototype structure is to be adjusted to form three asymmetric structures, which have the common experimental BRB frame on Frame Line B. The objective in Phase II is to verify the validity of the η-MPA procedure to estimate the seismic demands of asymmetric structures. This experiment also provides great opportunities to further enhance the networked pseudo-dynamic test and data archiving techniques envisioned for the Internet-based Simulations for Earthquake Engineering (ISEE) (Wang et al., 2005) launched in 2002 in Taiwan. This paper illustrates the analytical predictions computed from a general purpose frame response analysis program, PISA3D (Tsai and Lin, 2003) and evaluates the demands imposed on the experimental facilities and the experimental specimen.

The prototype building was first designed according to the story force distribution prescribed in the 2002 Taiwan Seismic Building Specifications (ABRI, 2002). The beam-to-column joints of the perimeter frame are all moment connections and all the other beam-to-column joints are pin-connected. A double-core BRB (Uang et al. 2004), with cement mortar infilled in two rectangular tubes, has been installed in each story of the BRBF specimen (Fig. 1a). All beams and columns are wide flange sections. The prototype 2-story building structure is assumed to be located in Chiayi City with Soil Type I (hard rock site) and the occupancy importance factor I is 1.0. The design dead load (DL) and live load (LL) are 6.89kN/m^2 and 2.45kN/m^2, respectively. The steel beams at the perimeter frame are A36 and all other members are A572 Grade 50. Design load combinations include: (1) 1.2DL+0.5LL+1.0EQ, (2) 0.9DL+1.0EQ, and (3) 1.2DL+1.6LL. It should be noted that the design spectra for the 10/50 event (Fig. 3a) are S_{DS}=0.8g and S_{D1} =0.45g, while for the 2/50 event (Fig. 3b), S_{MS}=1.0g and S_{M1} =0.55g. The members sized from the force-based method were compared with those determined from the displacement based procedures described hereafter.

DISPLACEMENT-BASED SEISMIC DESIGN PROCEDURE

The design procedures adopted for the prototype building consist of the following steps: 1) select an initial desired displaced shape for the structure, 2) determine the effective displacement by translating the actual MDOF structure to the substituted SDOF structure, 3) estimate system ductility from the properties of BRB members, 4) determine the effective period of the substituted SDOF structure from an inelastic design displacement spectra, 5) compute the effective mass, effective stiffness, and design base shear, 6) Distribute the design base shear over the frame height, 7) design the members for the steel frame. There are some key points in these steps described above. First, the i^{th} yield story drift θ_{yi} corresponds to the brace yielding can be estimated as:

$$\frac{\varepsilon_c}{\varepsilon_{wp}} = \frac{1}{\dfrac{L_j A_c}{L_{wp} A_j} + \dfrac{L_t A_c}{L_{wp} A_t} + \alpha_c} = \gamma \tag{1}$$

$$\theta_{yi} = 2 \cdot \varepsilon_{cy} / \gamma \cdot \sin 2\phi \tag{2}$$

where ε_{cy} is the yielding strain of the brace center cross section, γ is the ratio between a specific elastic axial strain of the brace center segment and the corresponding elastic averaged strain of the entire brace ε_{wp} (computed from the brace end work-point to work-point length). ϕ is the angle between the horizontal beam and the brace. θ is the story drift angle, ε_c is the strain of the brace steel core section and $\alpha_c = L_c/L_{wp}$ (L_c and L_{wp} are defined in Fig. 4). Thus, if θ_{mi} is the target drift of the i^{th} story calculated from the target displacement profile, then the story ductility can be computed from:

$$\mu_i = \theta_{mi} / \theta_{yi} \tag{3}$$

After calculating all the story ductilities from Eq. 3, the average of all story ductilities is taken as the system ductility. Since the BRBs are the primary energy dissipation element under the two levels of earthquakes, the connecting beams and the columns need to be designed considering the capacity design requirements. Typical force versus deformation relationships for A572 Gr.50 steel BRB specimens, is shown with actual yield capacity ($A_c \times F_{y,actual}$) in Fig. 5 (Tsai and Lin 2003). It is evident that the peak compressive force is slightly larger than the peak tensile force under large cyclic increasing strains. In addition, the strain hardening factor of Grade 50 steel is about 1.3 (for typical A36 steel, strain hardening factor can reach 1.5). Therefore, the maximum possible brace force can be estimated as follows:

$$P_{\max} = \beta \times \Omega \times \Omega_h \times P_y \tag{4}$$

where P_y is the nominal tensile yield strength, Ω accounts for possible material overstrength, Ω_h represents the effects of strain hardening, and β is about 1.1 considering the 10% difference between the peak compressive and tensile forces. Since the actual yield strength obtained from the tensile coupon tests will be employed to adjust the final BRB cross sectional area before fabrication, the material overstrength factor Ω is not included in the capacity design of members or connections for the BRBF specimen. Applying LRFD specifications (AISC, 1999):

$$P_u/(\phi_c P_n) \geq 0.2 \;:\; P_u/(\phi_c P_n) + \frac{8}{9} M_u / \left[\left(1 - \frac{P}{P_e} \right) \phi_b M_n \right] \leq 1.0 \tag{5}$$

where $P_n = F_{cr} A_g$, $P_{cr} = \pi^2 EI/(kl)^2$, $\phi_c = 0.75\ (tension)\ or\ 0.85 (compression)$, $\phi_b = 0.9$. Without considering the effects of the concrete slab but incorporating the spacing of the floor beams, unbraced length equal to 3.0 m is conservatively used to calculate the P_n and M_n in Eq. 5 for the capacity design of beam element framing into the braces.Two hazard levels considered in this study, for the 10/50 and 2/50 events, the inter-story drift limits are set at 0.02 and 0.025 radians, respectively. The actual material test results given in Table 1 can be used to refine the estimations of the ductility demand. With the actual steel core strength and assume the length ratio α_{ci} for braces at 1st- and 2nd- floor as 0.7 and 0.6, respectively, the averaged system ductility demands for the 10/50 and 2/50 events are 5.39 and 6.74, respectively (shown in Table 2). Applying the target displacement profile obtained in the Step 2, the effective displacement δ_{eff} is 0.13 m and 0.17 m for the 10/50 event and 2/50 event, respectively. Intersecting the effective target displacements of 0.13 m and 0.17 m on the inelastic displacement response spectra shown in Fig. 6, the effective first vibration period $(T_{eff})_1$ during the 10/50 and 2/50 events can be found as 1.19 and 1.22 second, respectively. In Fig. 6, the elastic displacement response spectrum (μ=1.0) is also given. Based on the results computed by the aforementioned Steps 5 and 6, the design base shears, 4149.8 kN (=0.24W) and 4509.4 kN (=0.26W) represent the stage of significant system yielding for the two events. It is evident that the 2/50-0.025 hazard/performance criteria govern the design. In the transverse direction (Y-direction), the stiffness ratio (SR) of the BRBF and the MRF is assumed 3, thus the BRBF resists 75% design earthquake force. Using this criterion, the core cross-sectional areas of the A572 Gr.50 steel BRB are 50cm^2 and 33cm^2 for the 1st- and 2nd-story, respectively. Using the capacity design principle, more than 14x24-mm diameter A490 bolts would be required for each first story BRB connection. In order to reduce the length of the connection, welded brace-to-gusset connection details are adopted. For the BRB in the second story, bolted details using 10-24mmϕ A490 bolts are adopted at each brace end. After a few iterations, the final dimensions of the braces and other members are selected as shown in Fig.7. In particular, the BRBs will yield in the proximity of the design story shear. The vibration periods are 0.69sec and 0.57sec in the longitudinal (MRF, noted as X-direction) and transverse (BRBF+MRF, noted as Y-direction) directions, respectively.

GROUND MOTIONS AND EXPERIMENTAL SCENARIO

According to the IBC2000 provisions (ICC, 2000) and the recommendations provided by Shome et al. (1998), for each pair of (bi-lateral) horizontal ground motion components, the square root of the sum of the square (SRSS) of the 5% damped site-specific spectra of the scaled horizontal components shall be constructed. The ground motions shall be scaled such that the spectral acceleration of the SRSS spectra is not less than 1.4 times the 5% damped smoothed design spectra at the fundamental period of the prototype building in the considered direction. In addition, the scaling factor (denoted as SF) should not exceed 4.0. Two pairs of earthquake records from the 1999 Chi-Chi earthquakes, CHY024 and TCU076 are used to be input time-histories. The corresponding spectra representing 10/50 and 2/50 hazard levels and satisfying the above mentioned requirements are shown in Fig. 3. The earthquake scenario, including the earthquake intensities and sequence, for Phase I of this experiment is shown in Fig. 8. Fig.9 shows three earthquake ground accelerations which are 50/50 (CHY024), 10/50 (TCU076), and 2/50 (CHY024) in the longitudinal and transverse directions. Before applying any pseudo-dynamic load, free vibration test will be conducted to evaluate the fundamental period and damping ratio of the entire prototype frame.

For the Phase II tests, in order to verify the η-MPA procedure (Chopra and Goel, 2004) for evaluating seismic demands for asymmetric-plan structures, the cross-sectional areas of the bracings of virtual BRBF on Frame Line D are intentionally reduced to one half of those of BRBF on Frame Line B. The properties of all the other members are the same as those in Phase I tests. It makes the building structure become asymmetric where center of mass (CM), center of stiffness (CR) and center of strength (CV) are no longer coincided as shown in Fig. 10, while the experimental specimen is still available for the Phase II tests. The centers of mass are located at the geometric center of each floor which is simulated as a rigid diaphragm. Moreover, the rotational inertia of the asymmetric structural model is varied, as 1.0, 1.8 and 3.0 times the rotational inertia, denoted as I_0, of the original symmetric structure. These are to represent the torsionally-stiff, torsionally-similarly-stiff and torsionally-flexible structures, respectively. The period and dominant motion of each mode for these three asymmetric structures are listed in Table 4.

The seismic ground accelerations considered in this verifying test is the east–west component of Loma Prieta earthquake, recorded at LP89g04 site (Fig. 11). The ground accelerations are scaled up to PGA=0.87g, making the spectral acceleration (Fig. 12) at period 0.621 second coincide with that of smoothed design spectrum for the 2/50 event. The period 0.621 second is the first mode period in transverse direction (Y Dir.) of torsionally-stiff structure. The scaling factor SF is equal to 2.1. The seismic ground accelerations are applied in the transverse direction only. Three separate pseudo-dynamic tests on torsionally-stiff, torsionally-similarly-stiff and torsionally-flexible structures are denoted as ASY1, ASY2 and ASY3, respectively. Before conducting these pseudo-dynamic tests, the story stiffness of BRBF will be checked and the residual displacements resulted from its previous tests will be removed as much as one can by pushing or pulling the BRBF specimen back to its original position.

NONLINEAR ANALYSIS AND SEISMIC DEMAND PREDICTIONS

Analytical Model

Nonlinear static and dynamic time-history analyses have been conducted using the PISA3D program (Tsai and Lin, 2003). In the PISA3D model, all beams, columns were modeled using the two-surface plastic strain hardening beam-column element. All BRBs were modeled using the two-surface plastic strain hardening truss element. The 2nd order effects developed in the gravity columns are also considered.

Phase I Tests

In Phase 1, as noted above, three pairs of earthquake ground accelerations scaled to three different PGAs are planned for the sub-structural PDT of the BRBF specimen. According to the predictions from the PISA3D nonlinear response history analysis (NLRHA), the distributions of the peak inter-story drifts under the applications of 50/50, 10/50 and 2/50 three earthquake loads are shown in Fig.13. It suggests that the analytical peak inter-story drifts of 1^{st}- and 2^{nd}-story in transverse direction are going to be about 0.017 and 0.025 radians, respectively. The design targets of inter-story drifts, 0.02 and 0.025 radians for the 10/50 and 2/50 events, are reached. The analytical distributions of the peak BRBF story shear force under the applications of 50/50, 10/50 and 2/50 three earthquake loads are shown in Fig. 14. In the transverse direction, the peak BRBF story shear forces are 1964 kN and 1795 kN for the 1^{st}- and 2^{nd}-story, respectively. And the peak BRBF story shear forces in the longitudinal direction are all less than 240 kN. The peak and residual floor displacements under the applications of 50/50, 10/50 and 2/50 three earthquake loads are shown in Fig. 15.

The nonlinear static η-MPA procedures, based on MPA procedures (Chopra and Goel, 2004), are described as follows. Firstly, conduct nonlinear pushover analysis by using modal force distributions to obtain the capacity curve. Secondly, idealize the capacity curve as a bilinear curve and calculate the structural yield strength R_y and base shear coefficient η,

$$\eta = R_y \,/\left(M_{eff} \cdot g \cdot PGA\right) \tag{6}$$

where M_{eff} is the effective modal mass, and g is the gravity acceleration. Perform a NLRHA on the SDOF of the force-deformation relationship of the bilinear curve. In this study, with the parameter η, Nonspec program (Mahin and Lin, 1983) was conveniently used to compute the nonlinear spectral deformation S_d. The peak roof displacement is calculated as $\Gamma_1 \times S_d$, where Γ_1 is the 1^{st} modal participation factor. Figs. 16 show the seismic deformation demands predicted by the η-MPA (incorporating the first mode only) procedures under the applications of 2/50 three earthquake load. The spectral deformation is at the intersection of the spectral demand curve corresponding to η and the capacity curve. The peak story displacements computed by η-MPA and NLRHA methods under the applications of 2/50 earthquake load are compared in Fig. 17.

Phase II Test

The prediction of the peak responses for torsionally-stiff (ASY1), torsionally-similarly-stiff (ASY2) and torsionally-flexible (ASY3) structures by response history analysis (RHA) and η-MPA procedure (condering first and second modes) are also shown in Fig. 18a, b and c, respectively. The peak corner translations and rotations of the second floor, computed by η-MPA procedures, are between 0.11~0.16m and 0.18%~0.33% radians, respectively. On the other hand, the peak corner translations and rotations of the second floor, computed by RHA procedures, are between 0.13m and 0.12%~0.14% radians, respectively. From these analyses, it is evident that the peak translational responses could be more accurately predicted than the peak rotational responses. Moreover, the errors in predicting the peak rotations of second floor are equal to 83%, 136% and 50% for ASY1, ASY2 and ASY3, respectively. This and other (Chopra and Goel, 2004) studies seem to suggest that the η-MPA procedure still needs much improvement, especially for those structures which are highly coupled in translational and rotational responses.

SUMMARY AND CONCLUSIONS

Based on these analyses, summary and conclusions are made as follows:

1. The peak story drift is likely to reach 0.025 radian in the transverse direction in Phase I tests after applying the 2/50 design earthquake on the BRBF specimen. Analysis suggests that the DSD procedure adopted in the design of the specimen is effective in limiting the ultimate story drift under the design earthquake.
2. The peak story drift reached 0.015 radian in the longitudinal building (BRBF's out-of-plane) direction in Phase I tests. It appears that this moderate out-of-plane deformational demands will be imposed on BRB frame under the bi-directional earthquake loads.
3. Three and two actuators of 980kN at the first and second floor, respectively, in the BRBF's in plane direction should be adequate for the PDTs. So the horizontal actuators setup shown in Fig.19a and b for the 1^{st}- and 2^{nd}-floor is proposed.
4. According to other study the analyses made for Phase II tests, it appears that the MPA procedure still need improvement, especially for those structures which are highly coupled in translational and rotational responses.

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Table 1 Material test results

		Positions of Sampling		f_y (MPa)	f_u (MPa)
A572 Gr.50	2FL	BRB2	core steel	367.9	523.9
	1FL	BRB1	core steel	371.8	512.1

Table 2 Computation of story ductility and system ductility

Story	α_{ci}	Story Ductility					
		10/50			2/50		
		θ_{yi}	θ_{mi}	μ_i	θ_{yi}	θ_{mi}	μ_i
		unit : 1/1000 rad			unit : 1/1000 rad		
2F	0.70	3.49	20	5.73	3.49	25	7.16
1F	0.60	3.93	20	5.08	3.93	25	6.35
Average		3.71	20	5.39	3.71	25	6.74

Table 3 The member sizes and properties of BRB

	α_{ci}	A_c (mm^2)	A_t (mm^2)	A_j (mm^2)	L_c (mm)	L_t (mm)	L_j (mm)	L_{wp} (mm)	γ
BRB1	0.6	5000	8100	11200	6201	520	2081	8802	1.18
BRB2	0.7	3300	6226	9152	5396	450	3101	8947	1.33

Table 4 Properties of each mode

		1st Mode	2nd Mode	3rd Mode	4th Mode	5th Mode	6th Mode
Torsionally-stiff	Dominant Motion	X	Y	R	Y	X	R
	Period (sec)	0.688	0.621	0.453	0.205	0.204	0.143
Torsionally-similarly-Stiff	Dominant Motion	X	Y	R	Y	X	R
	Period (sec)	0.688	0.644	0.586	0.207	0.204	0.190
Torsionally-flexible	Dominant Motion	R	X	Y	R	X	Y
	Period (sec)	0.797	0.688	0.611	0.250	0.204	0.204

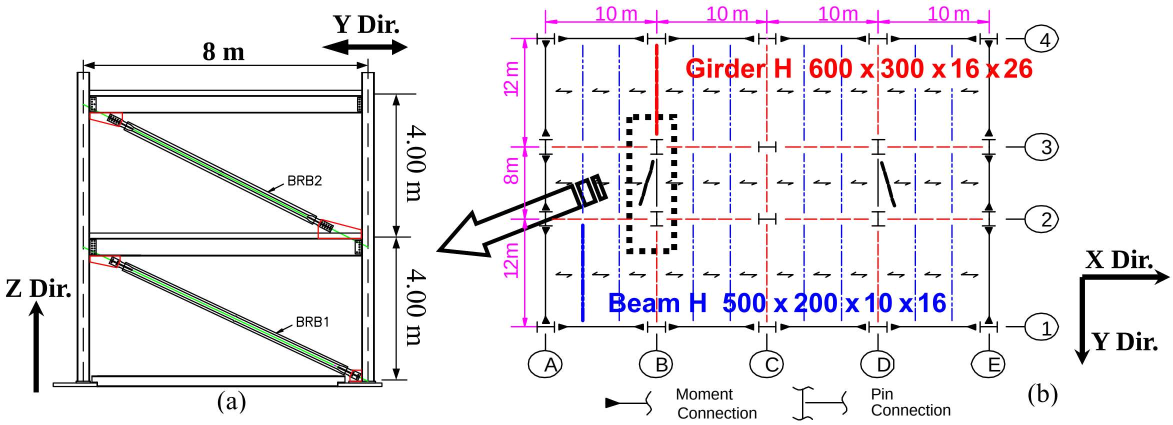


Fig.1 Sub-structural BRBF specimen elevation and floor framing plan of the prototype 3D steel frame

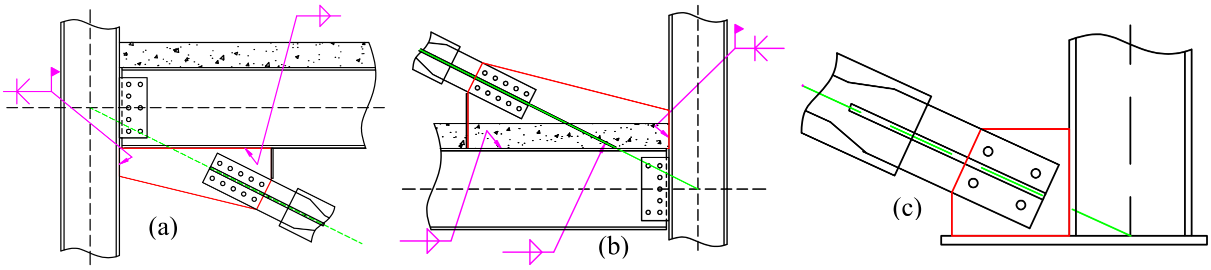


Fig.2 Details at the gusset plate connections

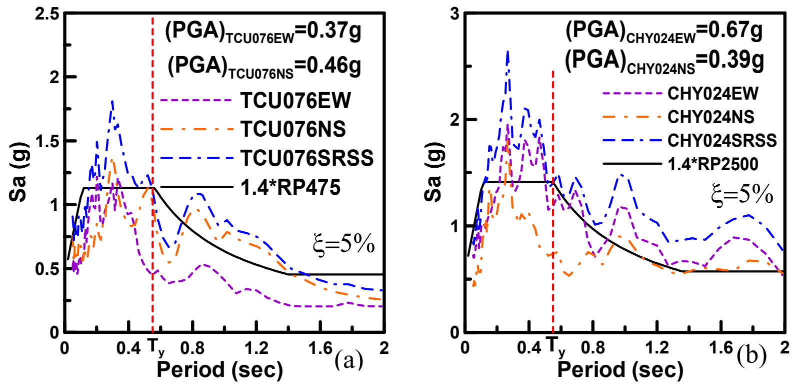


Fig. 3 Design acceleration spectra (a)10/50 (b)2/50 hazard level in Phase 1 PDTs

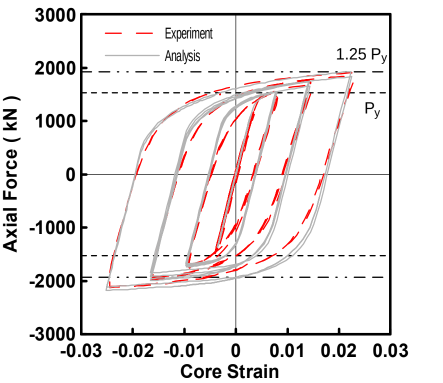


Fig.5 Force versus Core Strain Relationship of BRB using A572 Gr.50

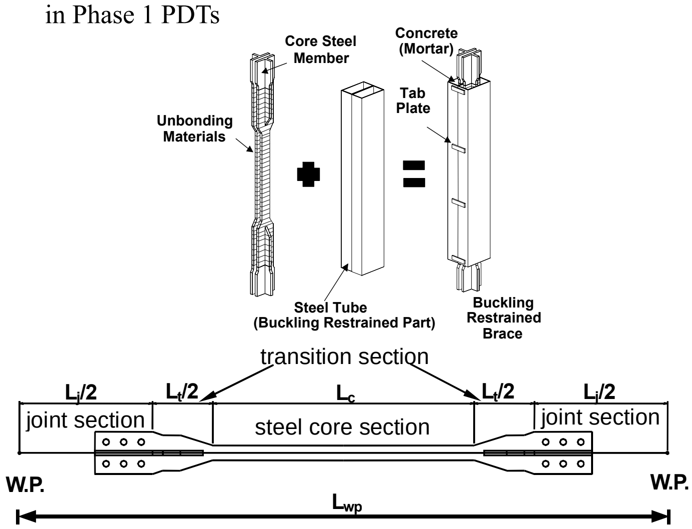


Fig.4 Profiles of core steel in the BRB

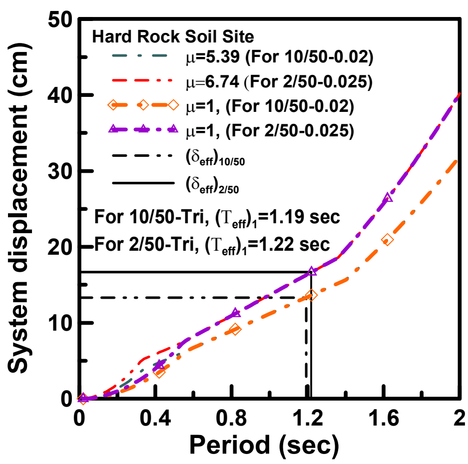
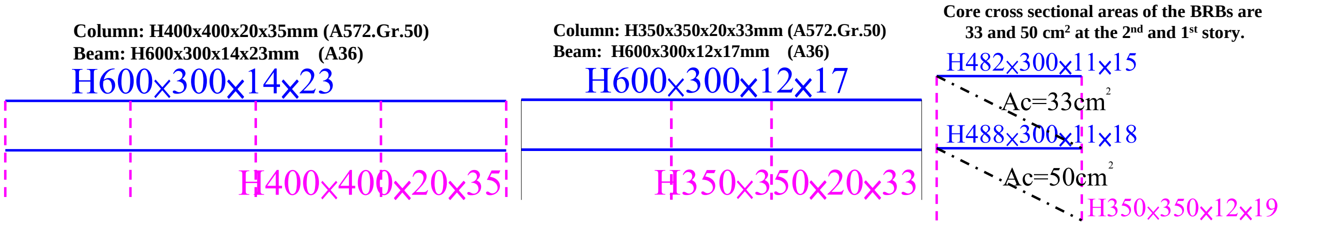


Fig.6 Inelastic Design Displacement Spectra



(a) Frame Elevation – Frame Line 1 and 4 (b) Frame Elevation – Frame Line A and E (c) Frame Elevation – Frame B and D

Fig. 7. Beam and column sizes in the MRF and BRBF

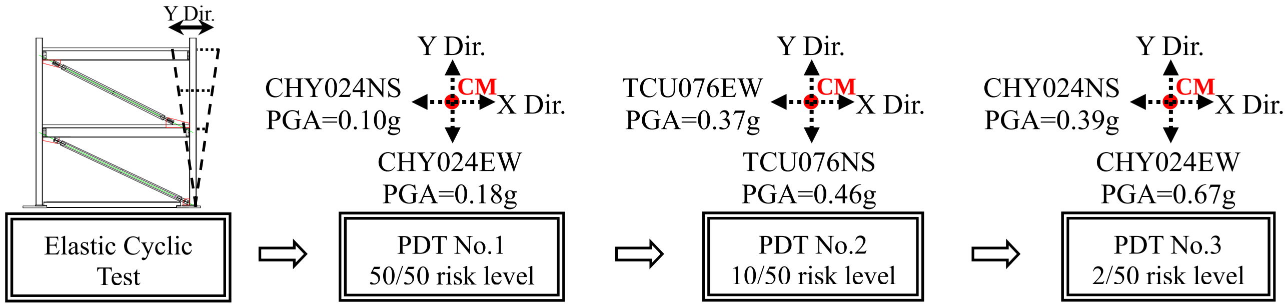


Fig.8 Earthquake scenario for Phase I of test

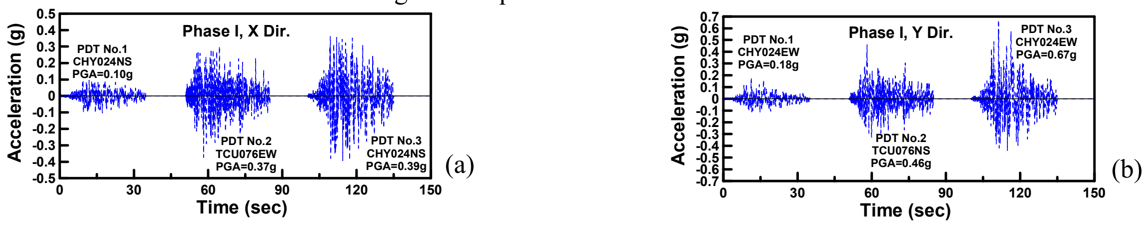


Fig.9 Ground acceleration time history involving three earthquake events for Phase I of test (a) X Dir. (b) Y Dir.

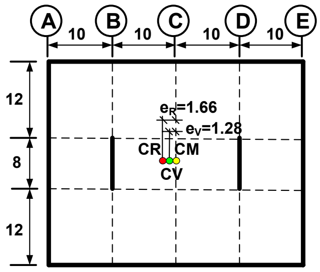


Fig.10 Locations of CV, CR and CM

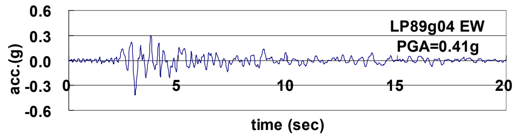


Fig.11 Seismic ground accelerations

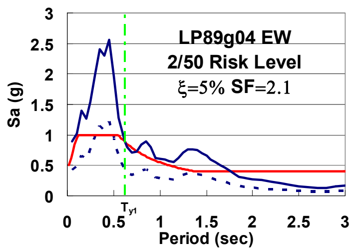


Fig.12 Acceleration spectra

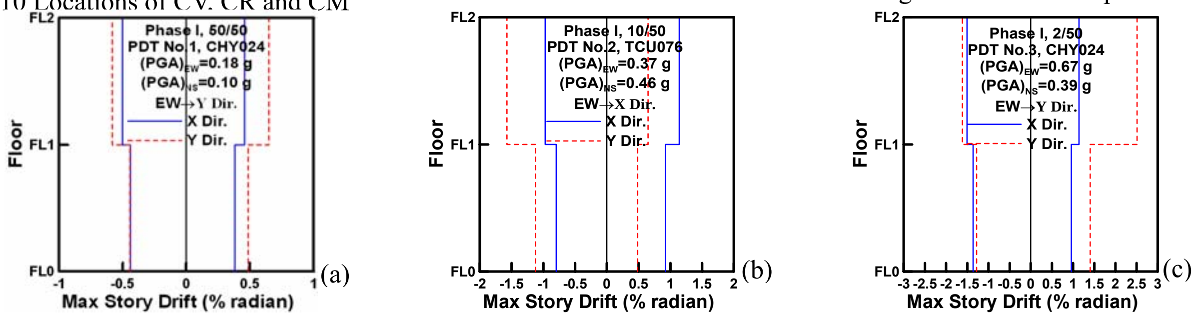


Fig.13 Peak inter-story drift distribution of BRB frame specimen (a)50/50(b)10/50(c)2/50 hazard levels

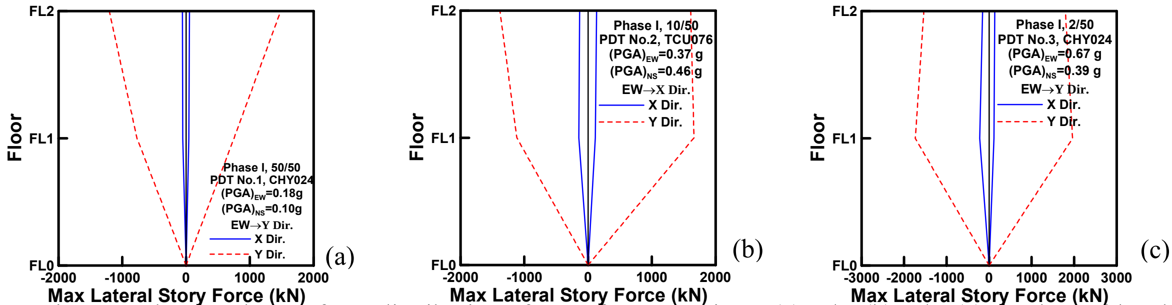


Fig.14 Peak Lateral story force distribution of BRB frame specimen (a)50/50(b)10/50(c)2/50 hazard levels

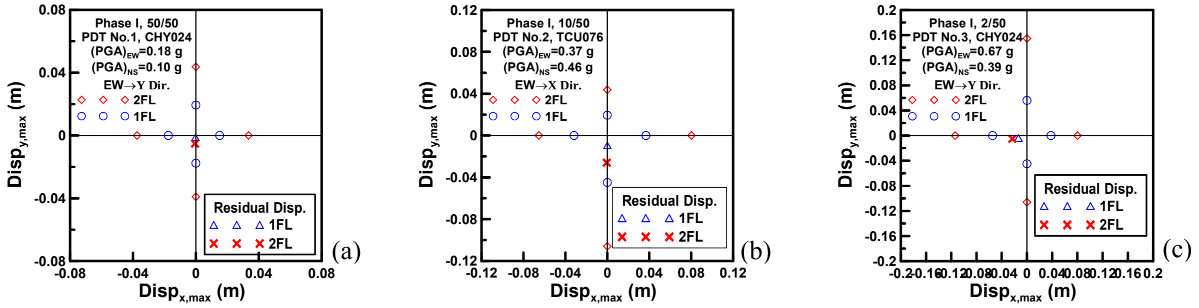


Fig.15 Peak and residual story displacements f BRB frame specimen (a)50/50(b)10/50(c)2/50 hazard levels

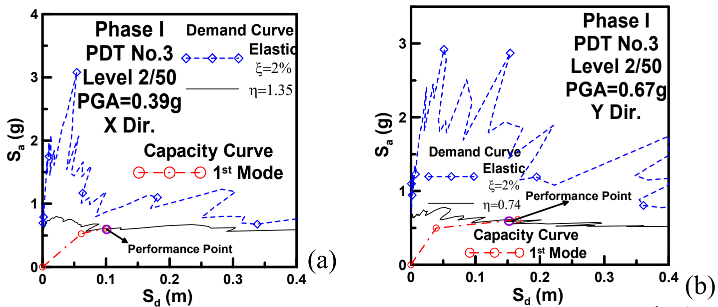


Fig.16 Displacement demands computed by η-MPA (a) X Dir. (b) Y Dir. in 2/50 hazard levels

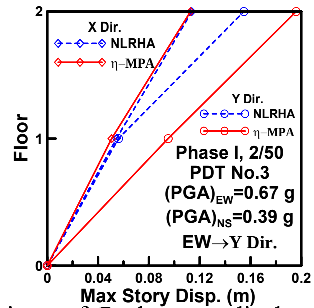


Fig.17 Comparison of Peak story displacement between the results computed by η-MPA and NLRHA method

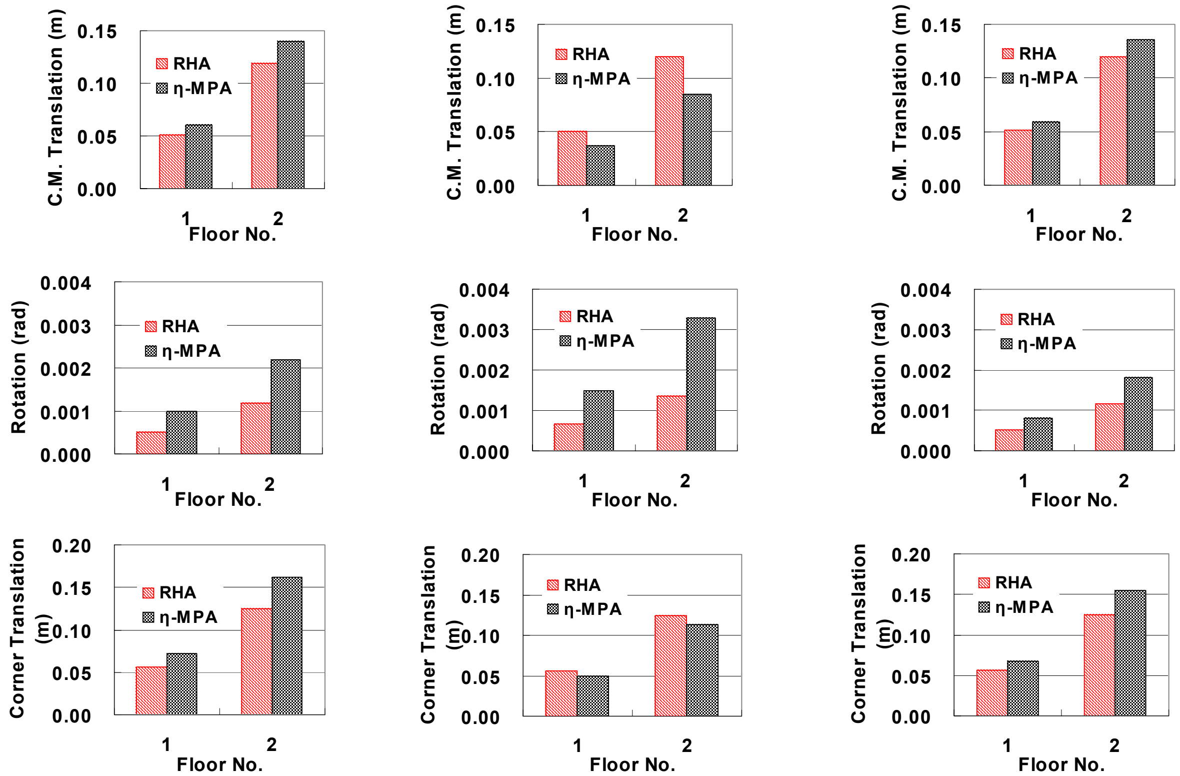


Fig.18 Prediction of peak responses for (a) ASY1 (b) ASY2. and (c) ASY3 by RHA and η-MPA procedure in Phase II

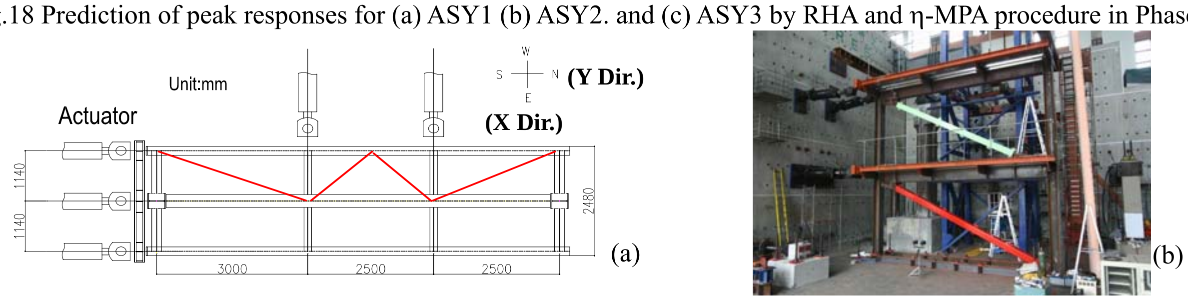


Fig. 19 (a) Experimental setup of horizontal actuators at first floor (b) Photo of sub-structural specimen