RESISTING CHARACTERISTICS OF HYBRID CENTER CORE SHEAR WALL SYSTEMS

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SUMMARY

In order to present the data of resisting characteristics of various kinds of hybrid center core shear wall systems in rigid frames for the antiseismic structural design, tests and analysis were carried out and compared directly with the corresponding moment frame systems for the estimation of antiseismic behaviours. Hybrid center core shear wall were made of steel panel with and without concrete covers. Steel profile and steel profile encased reinforced concrete composite moment frames were tested for the direct comparison of the resisting behaviours with the center core shear wall systems. Center core hybrid shear wall systems show 3 to 7 times higher initial stiffnesses and 2 to 3 times higher resistances than corresponding moment frames. However, the fracture ductility of center core hybrid shear wall systems show only very small fracture ductility 0.01 to 0.02 i.e. 0.1 of corresponding moment frames, and consequently very small energy dissipation capacity. Computed results coincide very well with test results. From the test results of 3 span 9 story rigid frames with and without hybrid center core shear wall systems it was shown very clearly the fact that center core shear walls are effective to increase their stiffness and resistances but cause the lack of ductility. This fact may enable to give the evaluation base of seismic performance of structures.

INTRODUCTION

There are yet few researches on steel panel shear wall [Mimura and Akiyama (1977), Elgaaly, Caccese and Du (1990, 1993), and Yamada (1992, 1996)]. Therefore there are very few researches on hybrid structure using steel panel shear wall. But the authors had proposed steel panel shear wall as one of the best antiseismic structural element [Yamada and Yamakaji, 1997]. Practically, it has already reported that center core steel panel shear wall set in the New Kobe City Hall of 32 stories of 132m heights showed good resisting behaviour subjected to severe ground motion at the last Kobe earthquake, 17. Jan. 1995, in Kobe, Japan [Yamada, 1996]. The hybrid structure to be proposed in this paper is structure in which both of steel frame and steel wall, or only frame is covered by concrete. The elasto-plastic deformation and fracture characteristics of each hybrid structure depends on the pattern of concrerte covering. On the basis of behaviours of infilled steel structures of simple material for comparison, the behaviours of hybrid structures may be extrapolated, so that those behaviours can be estimated by the application of foregoing method [Cardan, 1961].

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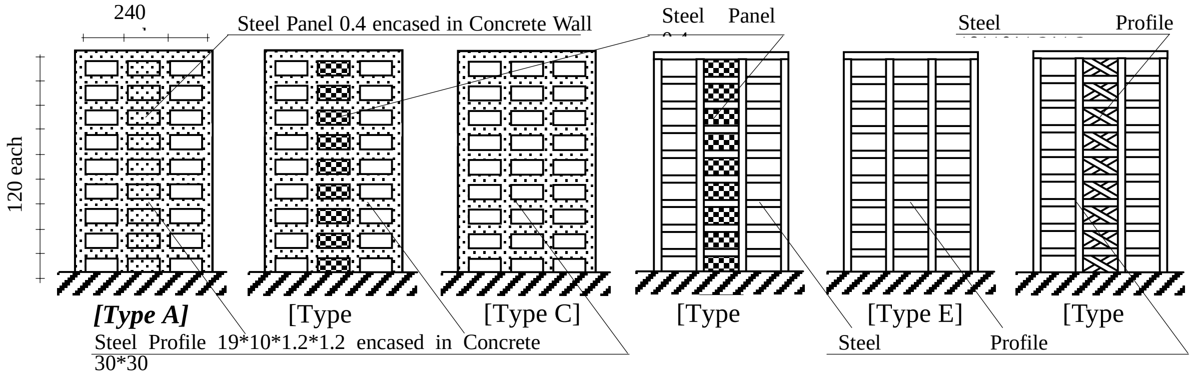


Figure 1 : Specimens (Unit : mm)

EXPERIMENT

Specimens

Tests were carried out on six specimens of 9 story 3 span with the same scale under the same loading condition. Six specimens are shown in Figure 1. Type A is steel profile encased reinforced concrete rigid frame with center core steel panel shear wall with concrete covering. Type B is steel profile encased reinforced concrete rigid frame with steel panel shear wall without concrete covering. Type C is steel profile encased reinforced concrete rigid frame. Type D is steel profile rigid frame with steel panel shear wall. Type E is steel profile rigid frame. Type F is steel profile rigid frame with center core steel X-shape brace with the same cross section. The mechanical properties of each material used for these six specimens are indicated in Table 1. Horizontal force P were loaded by incremental cyclic sway angles amplitudes of 0.001, 0.002, 0.003, 0.005, 0.007, 0.010, 0.015, 0.020, 0.030 and 0.100.

Table 1 : Material properties (Unit : MPa)

properties / specimens		compressive strength of concrete	yielding strength of steel profile	yielding strength of steel panel
Type	A	22.4	336	238
	B	42.5	336	238
	C	35.9	328	
	D		336	238
	E		328	
	F		328	

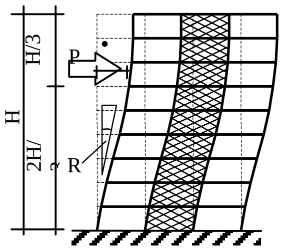


Figure 2 : Testing and Measuring method

Loading processes and Test results

Loading processes and test results are described as follows and are shown in Figure 3. Type A, shear wall in 1st to 6th story and the side columns cracked as one block. The crack pattern shows an inclination of 45 degree in 2nd to 6th story and, in 1st story, is similar to crack near fixed end like contilever. Type B formed diagonal tension field in buckled waves with a inclination of 45 degree as found in Type D. The buckled waves in Type D penetrated into its beams while the buckled waves in Type B is formed in each story independently. Type C shows remarkable incremental stiffness degradation due to concrete crack, development and so on after reversed loading. This characteristics is indicated in all of hybrid structure with concrete covering. Type C and Type E, without center core, shows more stable behaviour than the other structures with center core. All type structures show hysteresis loops symmetrical with regard to original point of P-R coodinates. The Comparison of skelton loop of each structure is shown in Figure 4. This diagram indicates that Type C and Type E have much more ductile than the structure with center core. Type A, B and D, horizontal stiffness and energy dissipation capacity increase in proportion to number of hybrid member. It can be assumed that the center core in Type B and Type D resist by the diagonal tension steel panel shear wall, that the center core in Type A resists by the conjugate diagonal tension and compression of steel panel shear wall and concrete shear wall, relatively and that the center core in Type F resists by the conjugate diagonal tension and compression of X-steel brace. From the results, the behaviours of hybrid structures of with concrete covering may be extrapolated and can be simplified into models adequately.

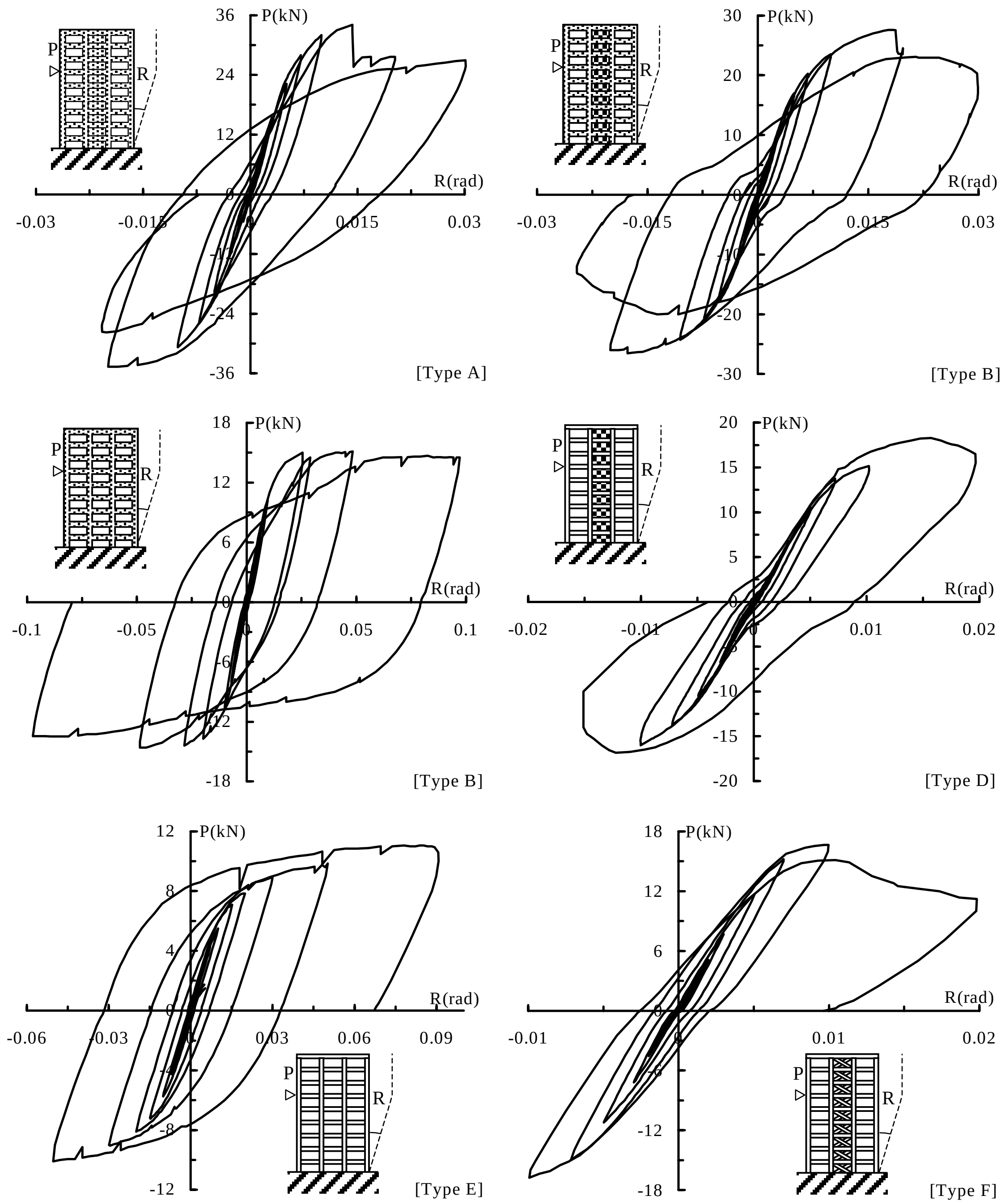


Figure 3 : Experimental P-R relationship of each structure

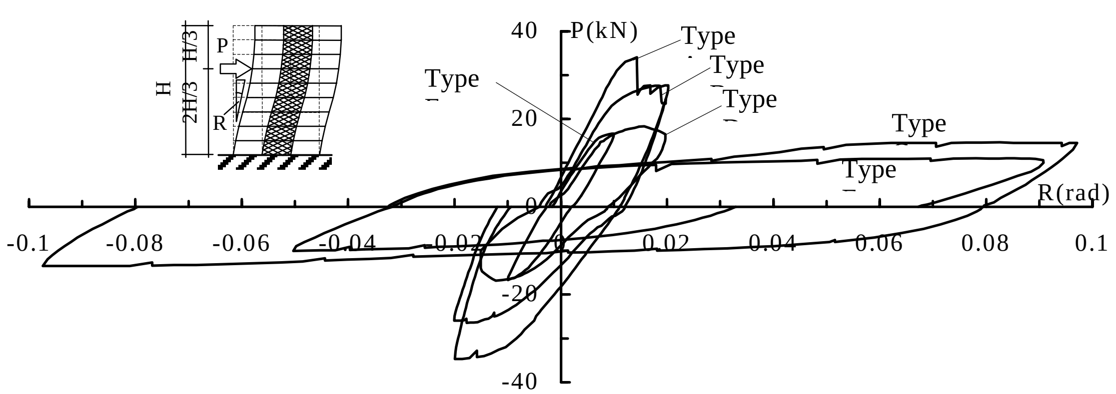


Figure 4 : Comparison of experimental envelopes of hysteresis loops

ANALYSIS

Models for Analysis

Horizontal sway behaviours of hybrid structures can be estimated by the application of foregoing method. Type A is simplified into a free-standing cantilever, such as shown in figure 5 (a) [Cardan, 1961], subjected to horizontal load of external excitation, horizontal and bending reaction by surrounding frame at each story and beams adjacent to center core [Yamada, 1980]. The structural elements of center core are 2^{nd} to 9^{th} walls differentiated in each story and the couple of side columns, which are deformed by sway and rotation around the center of gravity of 1^{st} story-wall, and 1^{st} wall deformed by sway deformation and the couple of side columns deformed by sway and bending deformation. This model is named Core wall system. While Type B and Type C are simplified into rigid members and hinges for rigid frame [Yamada and Kawamura et al., 1984] and shear spring for steel panel shear wall at each story, as shown in Figure 5 (b). For Type B, the stiffness of shear spring at each story can be put at arbitrary numerical values greater than zero. For Type C, the stiffness of shear spring at each story is put at zero as initial condition. In general, this model for Type B and Type C is named Frame system model.

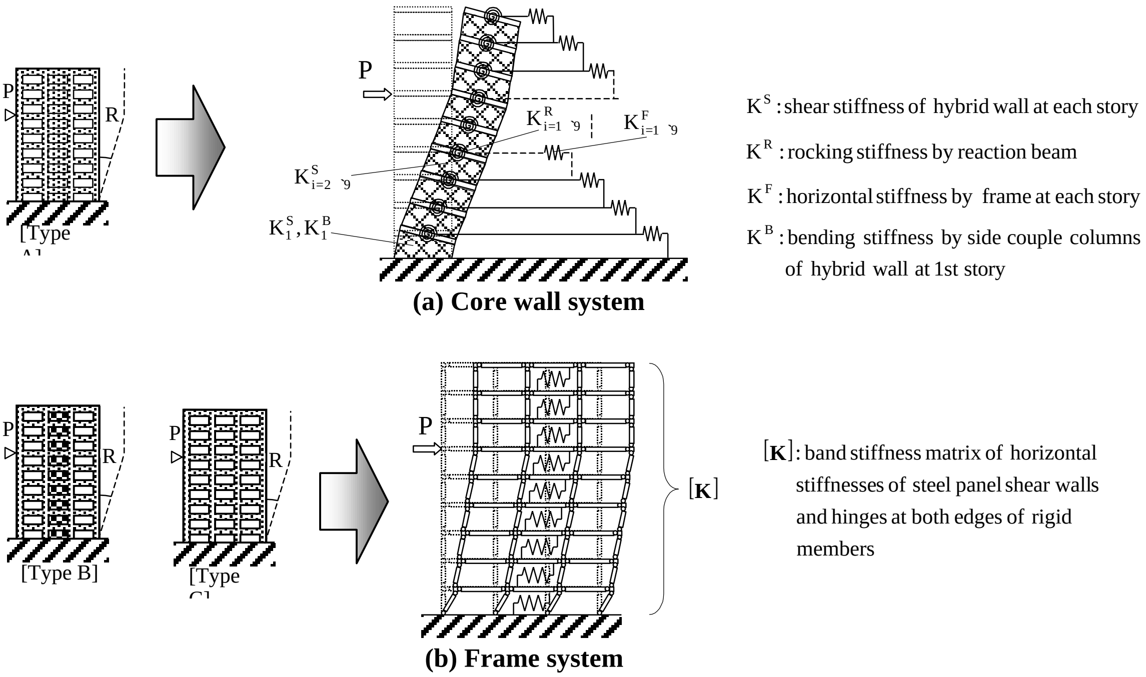


Figure 5 : Modeling of structures

Fundamental Equations

For core wall system, the incremental equation (1) that is derived from the shear equation at each story and the moment equation of the cantilever in the center of height at the 1^{st} story is given as follows:

$$\Delta\theta_1^B = \frac{\frac{1}{2}\left(11 - \frac{K_1^F}{K_1^S + K_1^F}\right) - \sum_{i=2}^{6} \frac{K_i^F}{K_i^S + K_i^F}}{\frac{\sum K_i^R + K_1^B}{h} + \frac{K_1^S \cdot K_1^F}{4\left(K_1^S + K_1^F\right)} + \sum_{i=2}^{9} \frac{K_i^S \cdot K_i^F}{K_i^S + K_i^F}} \cdot \Delta P \qquad (1)$$

where K^F and K^R are the horizontal spring stiffness by surrounding rigid frame reaction at each story and the rocking stiffness by bending reaction of beams adjacent to center core, respectively. K^S and K_1^B are shear stiffness of shear wall at each story and bending stiffness by the couple of side columns at the 1^{st} story, respectively. $\Delta\theta_1^B$ and ΔP are incremental bending angle of 1^{st} story of the couple of side columns and shear load, respectively. For frame system, the matrix equation (2) that is derived from the differential equation among the general story, the upper story and lower story is given as follows:

$$\{\Delta \mathbf{R}\} = [\mathbf{K}]^{-1}\{\Delta \mathbf{P}\} \qquad (2)$$

where $\{\Delta \mathbf{R}\}$ and $\{\Delta \mathbf{P}\}$ are the vector of incremental sway angle and shear load at each story, respectively. $[\mathbf{K}]$ is the band matrix, which is the horizontal stiffness of structure composed of rotation stiffness of hinges at both edges of rigid members and shear spring stiffness of steel panel shear walls at each story. For equation (1) and (2), incremental numerical analysis was carried out..

Structural elements

For core wall system with Type A, the relationship between shearing force and sway angle on each hybrid wall at 2nd to 9th story can be given as additional property in Figure 6 (a). On the basis of this relationship, hysteresis loop can be assumed mathematically. For simplified top and bottom of hybrid columns and both ends of hybrid beams with Type A, B and C, the relationship between bending moment and rotation angle can be given as degrading tri-linear hysteresis loop. Degraded inclination, which results from increase in concrete crack, development and so on experimentally, is simplified to aim at the last unloading point in having loaded in the same direction as the present in such a point that crack has closed. For Type C, Figure 6 (c) indicates that the mechanism and the assumed loop of steel panel shear wall. This diagram represents that, at the elastic stage, steel panel shear wall makes the formation of diagonal tension field in one direction after the shear buckling and come up to the yielding point of steel at last. And controlling point which describes the constant tension stress is decreased by given reversed deformation, and at last this tension field becomes compression field to buckle. From this stage, steel panel shear wall starts to make the formation of diagonal tension field in the conjugate direction and to lose the compression field at the same time. After the event, it is similar to the behaviour from the first stage. In solving equation (1) and (2) composed of stiffness of these assumed hysteresis, each time controlling points reach to boundary points, stiffness must be changed to continue to compute the other equation (1) and (2)

Analytical results

Figure 7 shows tested P-R relationship of compared with computed results of Type A, B and C. Type A, analysis underestimates the ultimate strength. Within about the sway angle of 0.010 at which cantilever yields at 1st story, computed values follow tested both qualitatively and quantitatively. Type B and C, computed values

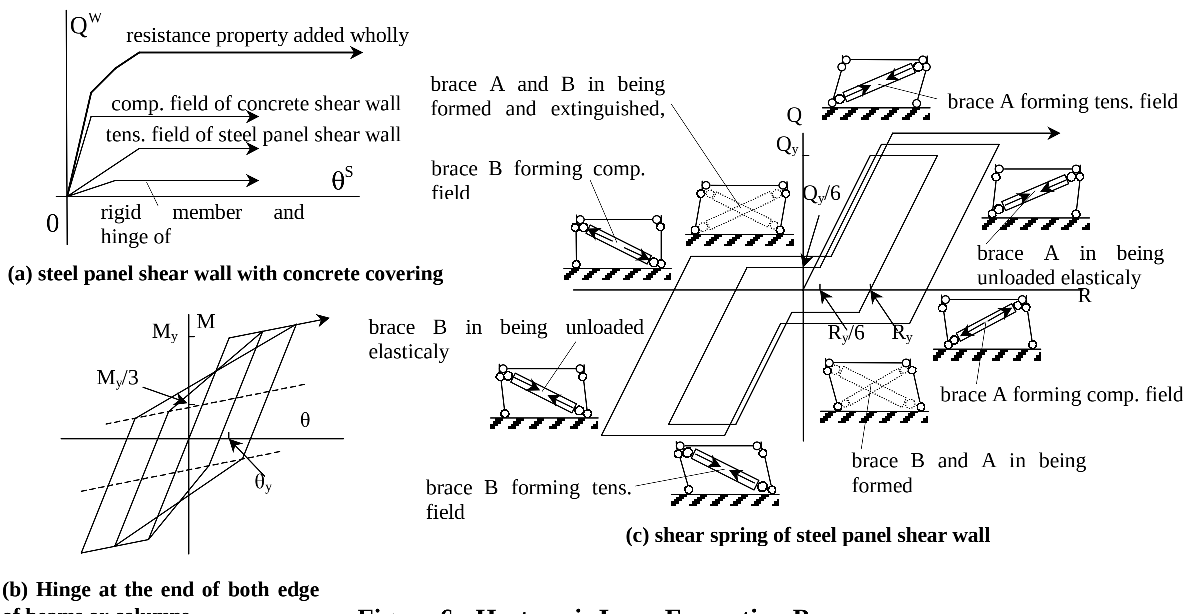


Figure 6 : Hysteresis Loop Formation Process

also coincide with tested value fairy good. Especially, non-linearity of whole behaviour due to change in system with steel panel shear wall and degrading-stiffness property with concrete is represented by analysis. And it is found that parallel damper of steel panel shear walls and serial damper of hinges each other in frame system model for Type B and C may influences structural performance. Figure 8 shows sway deflection curves with each model. Type A, horizontal displacement varies linearly throughout 1st to 9th story while Type B and C horizontal displacement varies with S-shape of inflection point in 2nd or 3rd story. Type A, B and C, sway deflection curves nearly symmetrical with the line of displacement of zero are presented at all. Figure 9 shows distribution of

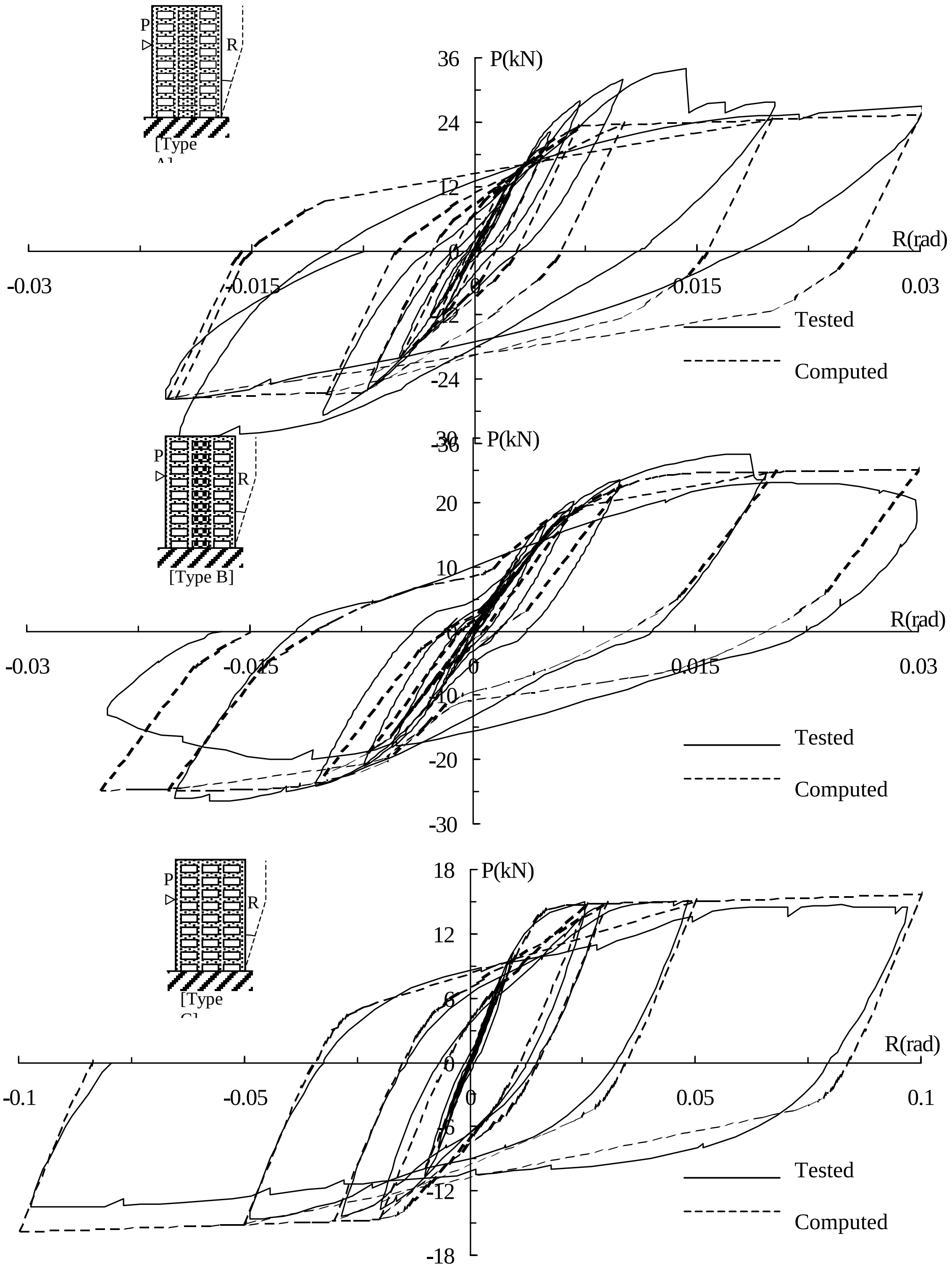
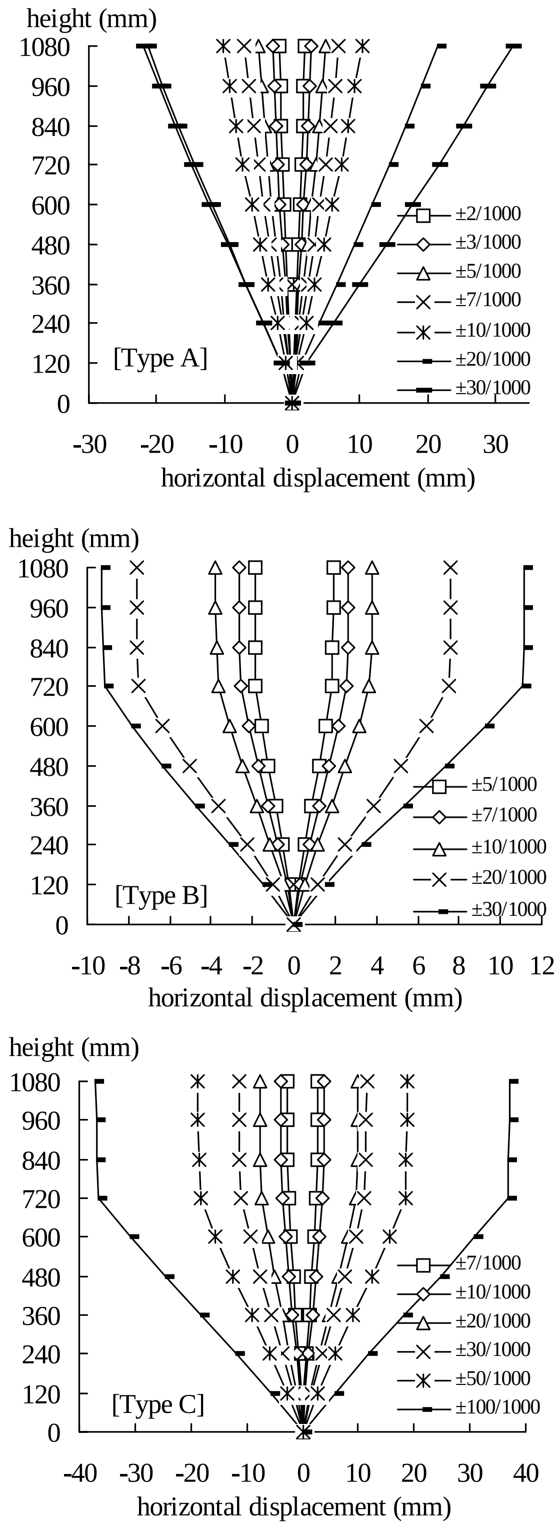


Figure 7: Comparison of P-R relationship between tested and computed values

story sway angle. Type A, thinking along with linearly deflection in Figure 8, bending deformation is larger than shear deformation at 1st story. Type B and C, it is found that shear deformation at 2nd or 3rd story is the largest of all stories. This indicate that steel panel shear wall in Type B starts to yield from the middle of story. And Type B, it is slightly found that forced deformation in one and the other direction gives reversing yielding turn of steel panel shear wall especially between at 1st and 6th story.



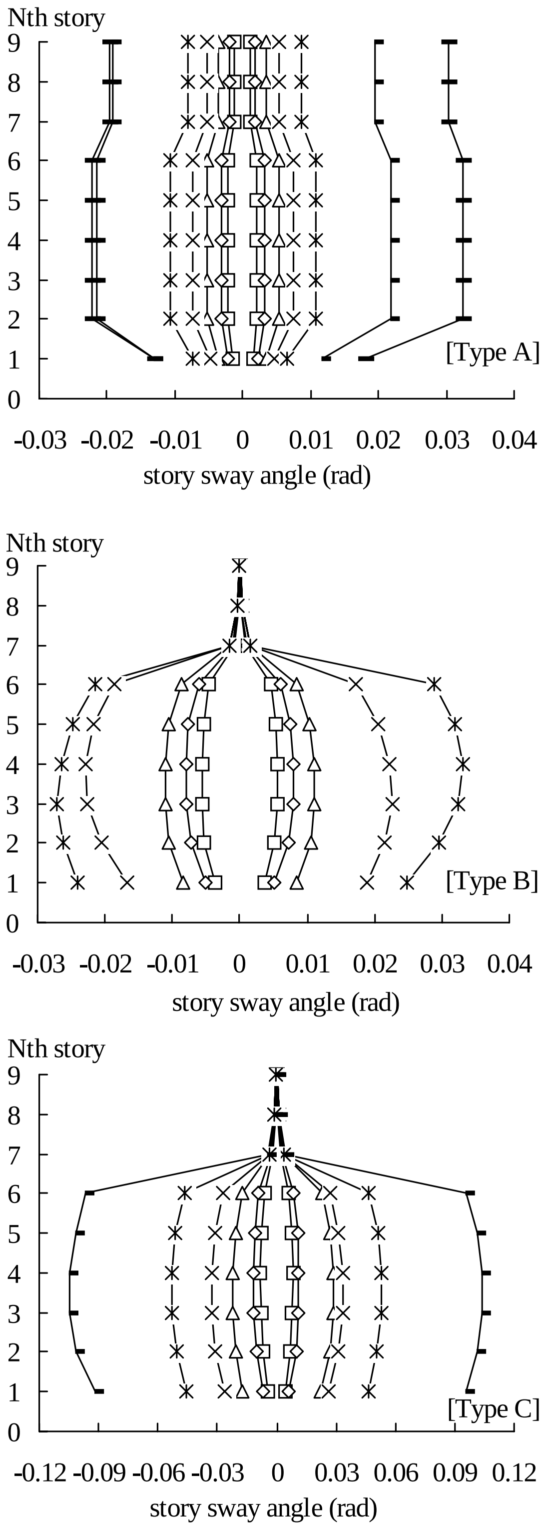


Figure 8 : Analytical deflection curves **Figure 9 : Analytical distribution of story sway angle**

CONCLUSION

In order to make clear the elasto-plastic deformation and fracture characteristics of hybrid center core shear wall system, tests and analysis were carried out. Hybrid structure Type A and B of steel panel shear wall with and without concrete covering, respectively, gives the lower deformation capacity and the higher initial stiffness and ultimate strength as shown in reinforced concrete structure, compared with corresponding moment frame. Experimental P-R relationship can be fairly well followed by incremental deformation method at which two simplified models are proposed.

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