Test Results of Four-Story Reinforced Concrete and Post-Tensioned Concrete Buildings: The 2010 E-Defense Shaking Table Test

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SUMMARY:
Two full-scale building structures were tested using a large shaking table facility, E-Defense. One was a four-story reinforced concrete building designed according to the present Japanese seismic design code, and the other was a four-story prestressed concrete building utilizing precast post-tensioned concrete members. Presented here are summaries of the global behaviors of both structures and key local damage and deformation observations. A comparison between the performances of the two structures is also presented.

Keywords: Reinforced concrete building, Post-tensioned concrete building, Shaking Table Test

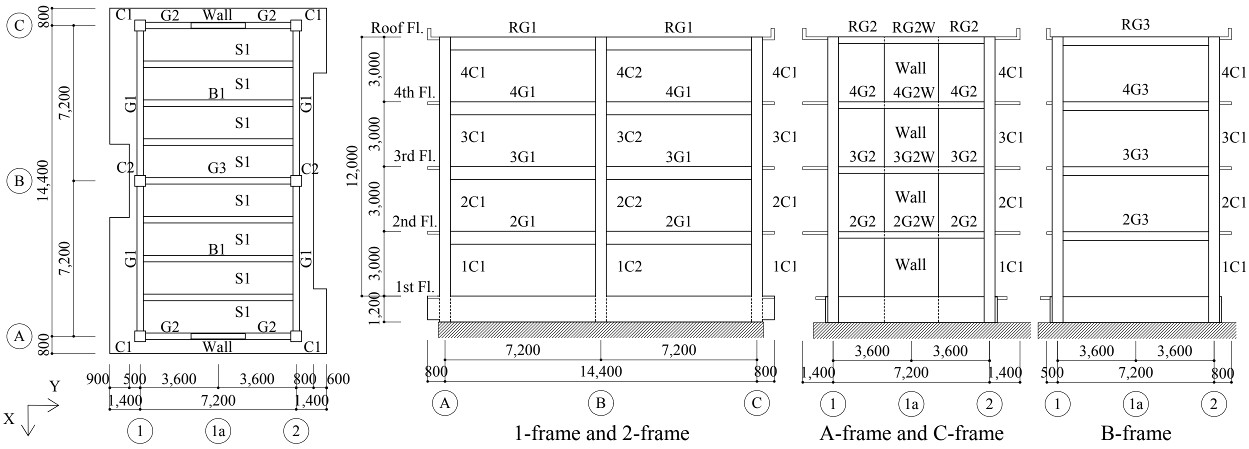
1. INTRODUCTION

Two full-scale building structures were tested using a large shaking table facility, E-Defense. One was a four-story reinforced concrete building (RC specimen) designed according to the present Japanese seismic design code, and the other was a four-story prestressed concrete building utilizing precast post-tensioned concrete members (PT specimen). Design and instrumentation of test structures were performed with input from the authors (Design and Instrumentation of the 2010 E-Defense Four-Story Reinforced Concrete and Post-Tensioned Concrete Buildings, PEER, 2011). Wherever possible, adjustments to the designs were made to bring the final structures closer to U.S. practice and benefit a broader audience.

2. TEST SPECIMENS

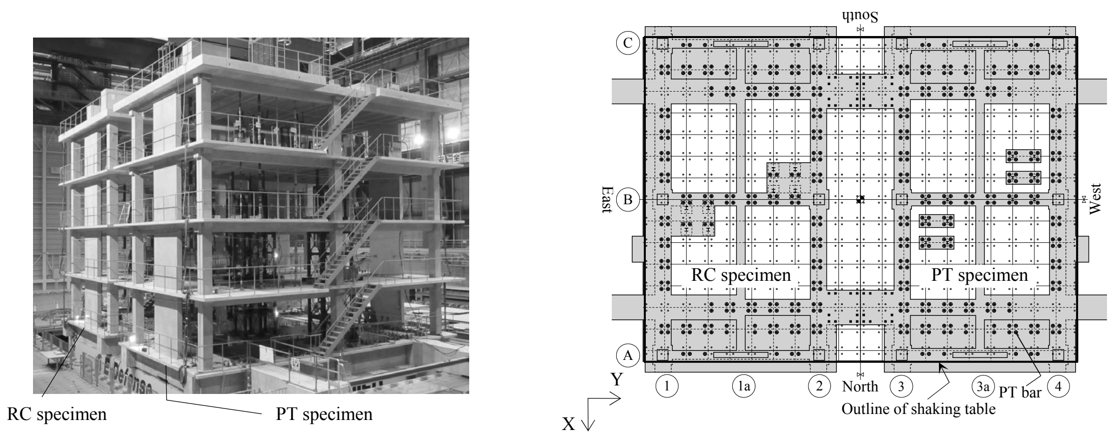
RC specimen and PT specimen having almost the same configurations and overall dimensions were prepared. The height of each story is 3 m. The moment frame system was adopted in the longer (X) direction of two spans of 7.2 m, and a pair of multi-story walls were incorporated in the exterior frames in the shorter (Y) direction of one span of 7.2 m. Figure 1 shows the plan and framing elevation of RC specimen. In this series of tests, RC specimen and PT specimen were set on the E-Defense shaking table and shaken simultaneously. Figure 2 shows the test setup of RC and PT specimens. The foundations were fixed on the shaking table by a number of prestressing post-tensioned bars. Column sections of RC specimen were 500 mm x 500 mm, and column sections of PT specimen were 450 mm x 450 mm. In the moment frame of X direction, the beam depth in RC specimen was 600 mm, and the beam depth in PT specimen was 500 mm. The wall section of both specimens was 250 mm x 2500 mm. Walls were coupled to the corner columns by beams with a depth of 300 mm. The thickness of the top slab was 130 mm and the foundation beam had a depth of 1200 mm. Strong steel frames were set in the specimens for collapse prevention and measurement of story deformations. Also, equipment was incorporated for additional research to assess the functionalities of building. The weights of the specimens were estimated based on the structural members, the fixed steel frames, and the equipment. The total weight of RC specimen was estimated at 5877 kN. The weights of the floors of RC specimen, from the second floor to the roof floor were 867 kN (2nd Fl.), 872 kN (3rd Fl.), 867 kN (4th Fl.), and 934 kN (Roof Fl.). The total weight of PT specimen was estimated at 5592 kN. The weights of the floors of PT specimen were 804 kN (2nd Fl.), 806 kN (3rd Fl.), 813 kN (4th Fl.), and 996 kN (Roof Fl.).

Figure 3 shows the reinforcement detail of RC specimen. RC specimen was designed in reference to the Japanese Building Standard for RC building. The concrete specified design strength was 27 N/mm^2. The longitudinal reinforcement was SD345 (nominal yield strength 345 N/mm^2) and the shear reinforcement was SD295 (nominal yield strength 295 N/mm^2). The amount of longitudinal reinforcement of beams resulted from allowable stress design. Longitudinal reinforcement of the



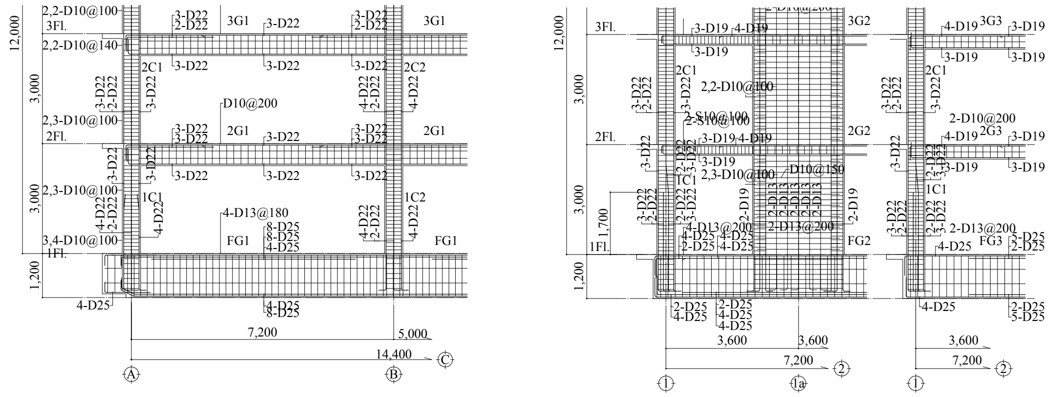
(a) Plan of 2nd to 4th floor (b) Framing elevation

Figure 1. Configuration of test specimen (RC specimen)



(a) 3/4 view of two specimens (b) Foundations fixed on shaking table

Figure 2. Test setup of RC and PT specimens



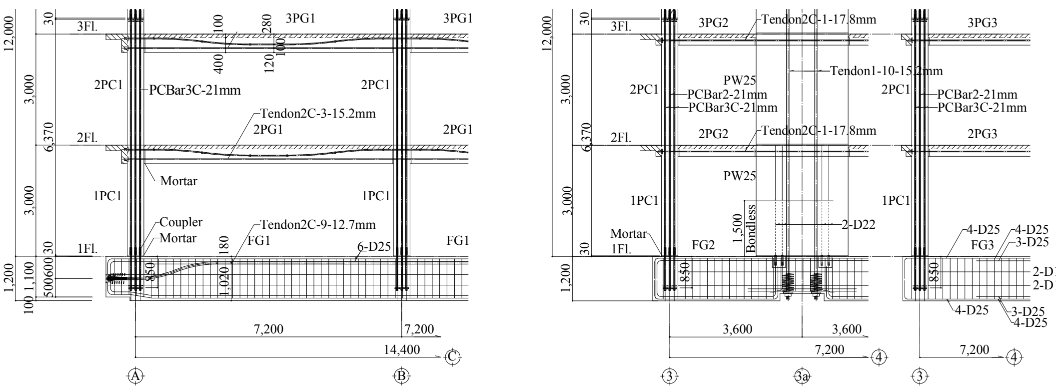
(a) X-direction frame (b) Y-direction frame

(Unit: mm)

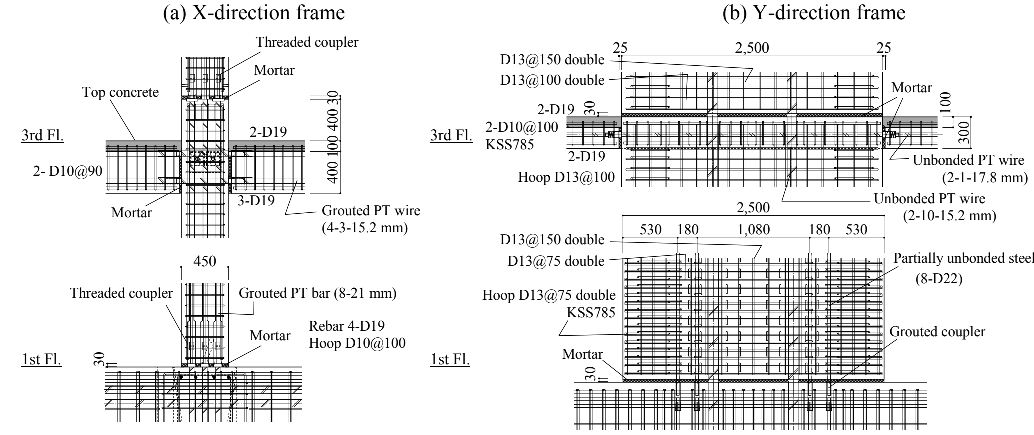
Figure 3. Reinforcement detail of RC specimen

columns was increased so that the column-beam strength ratios became larger than 1.0. The diameters of the longitudinal reinforcement of beams and columns were D22 [22 mm] and D19, while those of the walls were D19 and D13. The diameter of shear reinforcement was D10. When constructing RC specimen, columns, walls, beams, and the floor slab were cast together. The longitudinal reinforcements of columns, beams, and the end zones of walls were connected by gas pressure welding. Lap splices were used for the reinforcements of walls and floor slabs.

Figure 4 shows the reinforcement detail of PT specimen. Precast concrete columns, beams, and walls individually reinforced by longitudinal steel and shear reinforcements were assembled and connected by post-tensioned steel to generate the structure. Slabs were cast in place. The design strength of the precast concrete members was 60 N/mm^2. The design strength of the mortar for the grout bed at the ends of members was also 60 N/mm^2. The milk cement used to grout PT ducts had a design strength of



(a) X-direction frame (b) Y-direction frame



(c) Interior column (d) Wall

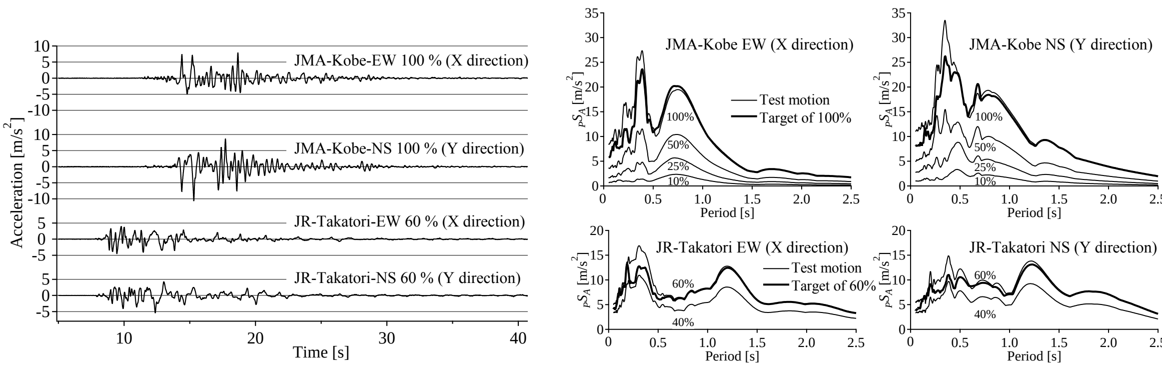
(Unit: mm)

Figure 4. Reinforcement detail of PT specimen

30 N/mm^2. The nominal strength of the PT bars was 1080 N/mm^2, while the nominal strength of the PT wires was about 1600 N/mm^2. The moment framing system adopted in X direction was designed in reference to the Japanese Building Standard for PT building. Columns were reinforced by longitudinal post-tensioned (PT) bars, and beams were reinforced by longitudinal PT wires. The ducts of columns and beams were grouted with milk cement after post-tensioning. A precast concrete slab was first attached to the beams and a concrete topping slab 100 mm thick was cast over it. The topping slab was connected to the beams through beam stirrups. The prestress level of the PT bars of the columns and PT wires of the beams in X direction was 80 % of the nominal yield strength. In Y direction, multi-story precast post-tensioned walls were incorporated in the exterior frames. Unbonded PT wires were used for the walls and beams of Y direction. The individual wall panels of each story were reinforced by longitudinal deformed steel bars and transverse deformed steel bars. Confining reinforcement was installed in the end regions of the walls. The unbonded PT wires extended from the base of the foundation to above the four wall panels, resulting in an unbonded length of about 12 m. At the interface between the wall base and foundation, partially unbonded deformed steel bars (8-D22) were used as energy dissipating elements. The prestress level of the unbonded PT wire of the walls and side coupling beams of Y direction was 60 % of nominal yield strength.

3. LOADING PROGRAM

JMA-Kobe and JR-Takatori recorded in the 1995 Hyogoken-Nanbu earthquake were adopted as the input ground motions. The NS-direction wave, EW-direction wave, and UD-direction wave were input to the Y direction, X direction and vertical direction of the specimen respectively. Time histories and acceleration response spectra for the input motions are presented in Figure 5. The intensity of input motions was gradually increased to observe the damage progress. The adopted factors for JMA-Kobe were 10 %, 25 %, 50 %, and 100 %. Following the JMA-Kobe motions, the JR-Takatori motion scaled to 40 % and 60 % was applied to impart large cyclic deformations. White-noise inputs were applied prior to each main test. The initial natural period of RC specimen was 0.43 sec in X direction and 0.31 sec in Y direction. The initial natural period of PT specimen was 0.45 sec in X direction and 0.29 sec in Y direction.



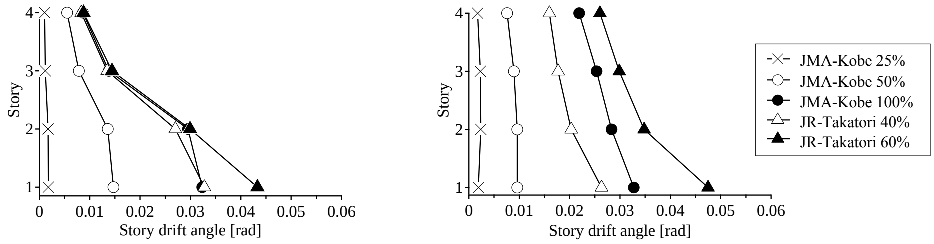
(a) Time histories

(b) Acceleration response spectra (Damping ratio= 5 %)

Figure 5. Input ground motion

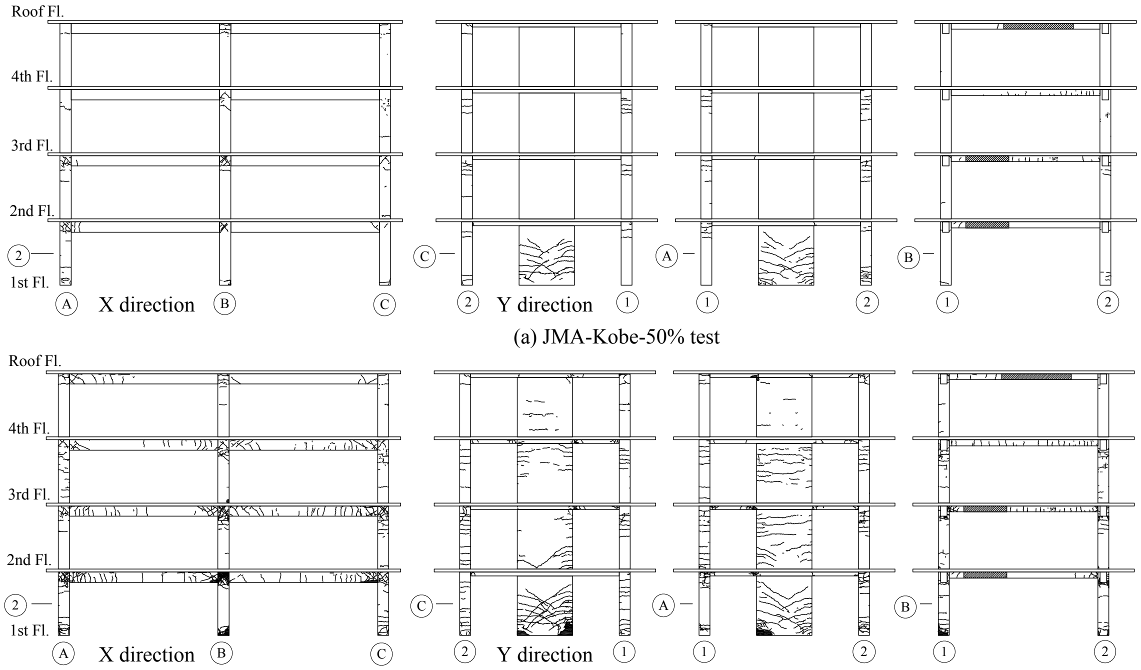
4. RESPONSE AND DAMAGE OF RC SPECIMEN

Figure 6 shows the distribution of maximum inter-story drifts over the height of RC specimen. Those were estimated at the center of floor. For the JMA-Kobe-25% test, the maximum inter-story drifts were smaller than 0.004 rad in X and Y directions. For the JMA-Kobe-50% test, the maximum inter-story drift in the first story was 0.015 rad in X direction and 0.010 rad in Y direction. For the JMA-Kobe-100% test, the maximum inter-story drift in the first story exceeded 0.03 rad in both X and Y directions. In X direction, the maximum inter-story drifts in the first and second stories were about twice that of the third story. In Y direction, the distribution was relatively uniform, although the



(a) X direction (b) Y direction

Figure 6. Maximum inter-story drift distribution of RC specimen



(a) JMA-Kobe-50% test

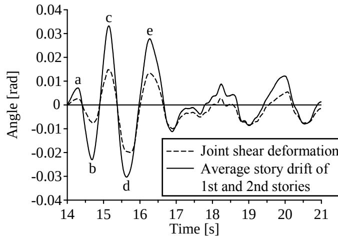
(b) JMA-Kobe-100% test

Figure 7. Overall damage states of RC specimen observed after JMA-Kobe-50% and JMA-Kobe-100% tests

maximum inter-story drift tended to grow in the first story. For the JR-Takatori-60% test, the maximum inter-story drift in the first story exceeded 0.04 rad in X and Y directions.

Figure 7 shows overall damage states of RC specimen observed after the tests. The beam-column joints, beam ends, column and wall bases showed minor cracking after the JMA-Kobe-50% test. Although the maximum measured shear crack width was 0.5 mm after the JMA-Kobe-50% test, the maximum shear crack width in interior beam-column joints reached 2.5 mm after the JMA-Kobe-100% test. The maximum shear crack widths at beam ends and exterior beam-column joints were limited to 1.1 mm after the JMA-Kobe-100% test. The cover concrete of first story column bases partially spalled to a height of 250 mm in the JMA-Kobe-100% test. The corner of first story wall bases suffered compressive failure to a height of 300 mm and width of 600 mm in the JMA-Kobe-100% test. The longitudinal reinforcement buckled in that region. Wall sliding at the base was observed in the JMA-Kobe-100% and subsequent tests.

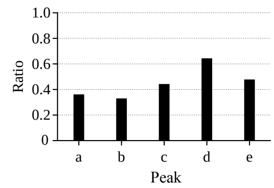
Shear deformation of the beam-column joints was measured in X direction. In the JMA-Kobe-100% test, the interior joint suffered severe damage at 2nd Fl. Figure 8 (a) shows the time history of the shear deformation angle of the 2nd Fl. joint, comparing to the average inter-story drift angle of the upper and lower stories. Peaks of the inter-story drift were noted as a to e in the histories. The joint shear deformation significantly increased at Peak-c and Peak-d due to the cyclic large inter-story drifts of about 0.03 rad. As shown in Figure 8 (b), the significant shear cracks were observed at Peak-d. Figure 8 (c) shows the shear deformation ratio corresponding to Peak-a to Peak-e. The shear deformation ratio was defined as the joint shear deformation angle divided by the average inter-story



(a) Time history of joint shear deformation angle at 2nd Fl.

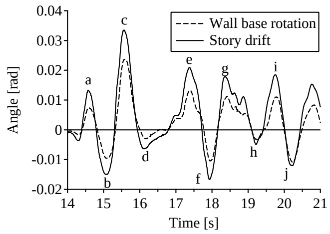
Peak-d

(b) Joint shear cracks at 2nd Fl.

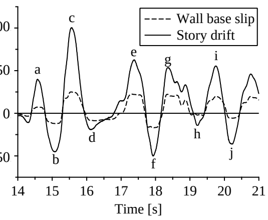


(c) Joint shear deformation ratio at 2nd Fl.

Figure 8. Local damage and deformation of RC interior beam-column joint in JMA-Kobe-100% test

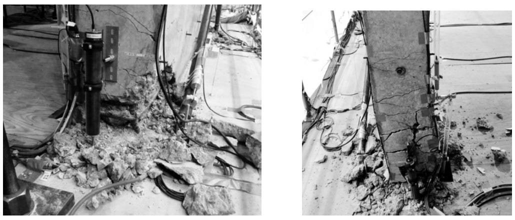


(a) Time history of wall base rotation

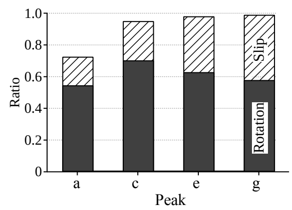


(b) Time history of wall base slip

(c) Condition of 1st story wall at Peak-c



(d) Conditions of wall base corners at end of JMA-Kobe-100% test



(e) Wall base deformation ratio

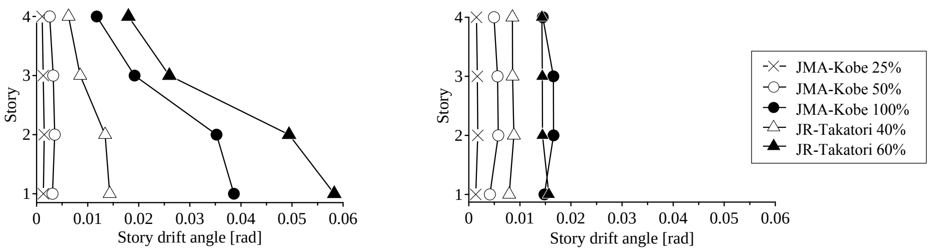
Figure 9. Local damage and deformation of RC wall base at C-axis for JMA-Kobe-100% test

drift angle. The deformation ratio was about 0.3 at Peak-a and increased to 0.6 at Peak-d.

Rotation and lateral slip deformations of the wall bases were measured in Y direction. Figures 9 (a) and (b) show the time histories of the wall base rotation and slip in the JMA-Kobe-100% test, comparing to the first story drift. Peaks of the inter-story drift were noted as a to g in the Figures. Figure 9 (c) shows the condition of the first story wall sustaining the large inter-story drift of 0.034 rad at Peak-c. A local compressive failure is seen at the base corner of A-side, and notable tension cracks due to the large inter-story drift are seen in the lower part of B-side. Figure 9 (d) shows the conditions of wall base corners at the end of JMA-Kobe-100% test. According to video observations, in the cycle when the inter-story drift approached Peak-d, the severe local buckling of bars occurred at the base of B-side due to the previous significant tension of bars at Peak-c. Figure 9 (e) shows the deformation ratios at the peak inter-story drift. The deformation ratios were defined as the ratio of wall base rotation and slip to the inter-story drift. At Peak-c, the inter-story drift was mostly derived from the rotation, with the deformation ratio of 0.7. Since the slip deformation becomes constant after the maximum deformation at Peak-c, the deformation ratio of the slip increased to 0.4 at Peak-g.

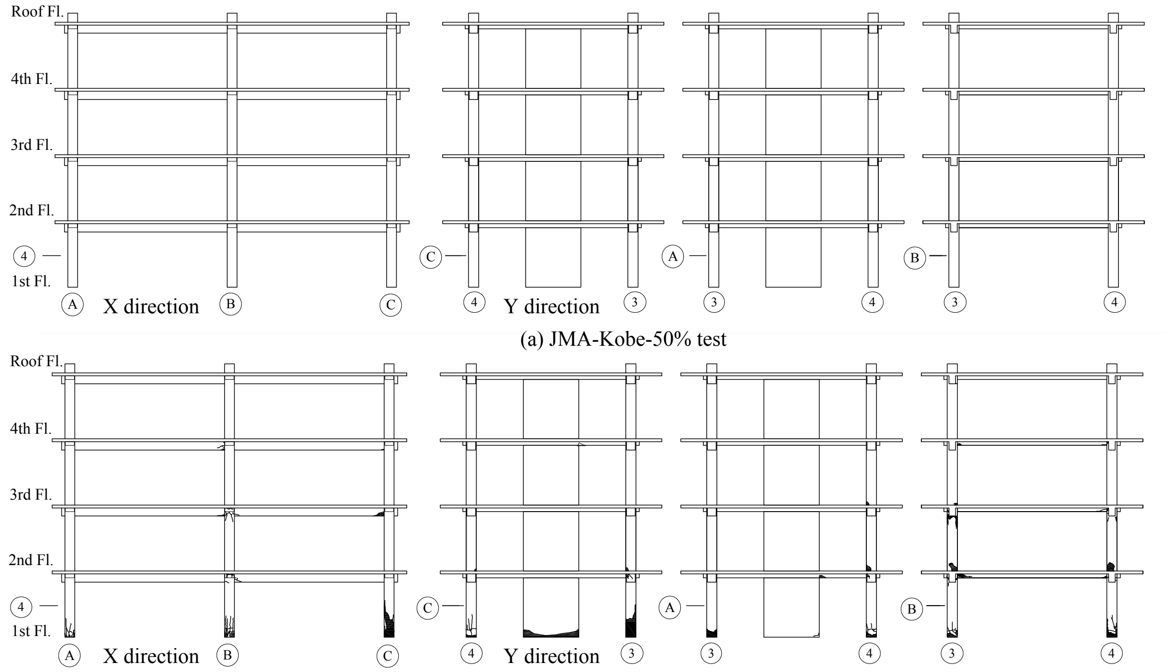
5. RESPONSE AND DAMAGE OF PT SPECIMEN

Figure 10 shows the distribution of maximum inter-story drifts over the height of PT specimen. Those were estimated at the center of floor. For the JMA-Kobe-25% test, the maximum inter-story drifts were smaller than 0.003 rad in X and Y directions. For the JMA-Kobe-50% test, the distributions were relatively uniform in X and Y directions, and the maximum inter-story drift in the second story was 0.004 rad in X direction and 0.006 rad in Y direction. For the JMA-Kobe-100% test, the maximum inter-story drift in the first story reached 0.039 rad in X direction, while the maximum inter-story drift



(a) X direction (b) Y direction

Figure 10. Maximum inter-story drift distribution of PT specimen



(a) JMA-Kobe-50% test

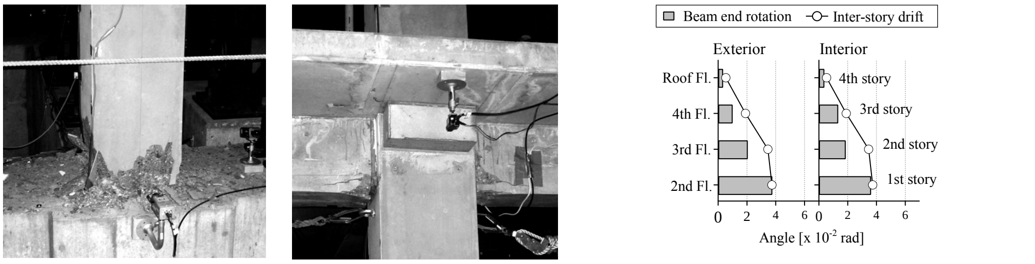
(b) JMA-Kobe-100% test

Figure 11. Overall damage states of PT specimen observed after JMA-Kobe-50% and JMA-Kobe-100% tests

in the first story only reached 0.015 rad in Y direction. In X direction, the maximum inter-story drifts in the first and second stories were about twice that of the third story, while in Y direction the distribution remained relatively uniform. For the JR-Takatori-60% test, the maximum inter-story drift in the first story was 0.058 rad in X and 0.015 rad in Y direction.

Figure 11 shows overall damage states of PT specimen observed after the tests. After the JMA-Kobe-50% test, no obvious damage was observed. After the JMA-Kobe-100% test, compressive failure of concrete due to large story drift of X direction was observed in beam ends and column bases. In X direction, minor shear cracks were also observed in the interior and exterior beam-column joints. The cover concrete of the first story wall base partially spalled to a height of 250 mm in the JMA-Kobe-100% test at C-axis. Note that wall panels and grout bases made of steel fiber reinforced concrete and steel fiber reinforced mortar were used in the first and second stories of A-axis but not of C-axis. For the JMA-Kobe-100% test, an obvious torsional motion was observed in Y direction, indicating the different degrading behaviors of the two walls. As a result, the wall at C-axis sustained larger deformations than that at A-axis.

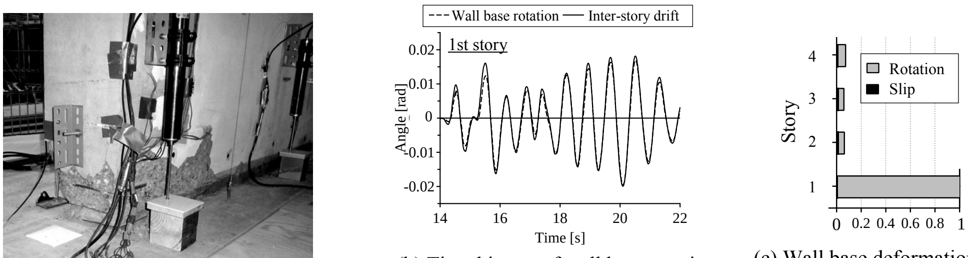
Figure 12 (a) shows the local damage to the PT moment frame at the end of JMA-Kobe-100% test. Just above the foundation, the interior column exhibits compressive failure with the cover concrete spalling up to a height equal to the sectional depth of the column. At the 2nd Fl. beam-column joint, the bottom corners of the beams exhibit compressive failure due to negative bending (top in tension). Rotation of the beam ends were measured in X direction. Figure 12 (b) shows the maximum beam end rotation angles and the maximum inter-story drift angles. In the JMA-Kobe-100% test, the maximum



(a) Interior column base at 1st Fl. and interior beam-column joint at 2nd Fl. at end of JMA-Kobe-100% test

(b) Maximum beam end rotation and inter-story drift in JMA-Kobe-100% test

Figure 12. Local damage and deformation in PT moment frame for JMA-Kobe-100% test



(a) Condition of 1st story wall base corner at end of JMA-Kobe-100% test

(b) Time history of wall base rotation in JMA-Kobe-100% test

(c) Wall base deformation ratio of maximum rotation and slip

Figure 13. Local damage and deformation of PT wall base at C-axis for JMA-Kobe-100% test

inter-story drifts exceeded 0.03 rad in the first and second stories. In the Figures, the beam end rotation angle at the 2nd Fl. is close to the first story drift angle, and the beam end rotation angle at the 3rd Fl. is half the second story drift angle. These measurements indicate that the top part of the columns in the second story yielded resulting in a partial mechanism of the first and second stories.

Figure 13 (a) shows the condition of the first story wall base corner at the end of JMA-Kobe-100% test. The core concrete in that region is adequately confined by lateral reinforcement. That region separated from the foundation in tension due to the overturning moment. Rotation and slip deformations of the wall bases were measured in Y direction. The time history of wall base rotation is shown in Figures 13 (b) and compared with the first story drift at C-axis. The wall base rotation and the first story drift are almost equal in the JMA-Kobe-100% test. Figure 13 (c) shows the deformation ratios, which are defined as the ratios of the maximum wall base rotation and slip to the maximum inter-story drift, for the first to fourth story. The deformation ratios of rotation are much smaller in upper stories than that in the first story, indicating that the majority of wall deformations is due to the rotation at the first story wall base. The deformation ratio of slip is close to zero in the first to fourth story.

6. COMPARISON BETWEEN RC AND PT SPECIMENS

RC and PT specimens sustained significant deformations in the JMA-Kobe-100% test. Figure 14 shows the relationship between base shear force and global drift angle for the JMA-Kobe-100% test, with the time history of base shear force. The base shear force was estimated from the horizontal inertia force. The horizontal inertia force was obtained as the measured acceleration times the floor mass. Global drift angle was defined as the fourth floor drift divided by its elevation. The base shear force and global drift angle were estimated at the center of floor. In the relationships between base shear force and global drift angle of the both specimens, the slope of the loading branch significantly decreased after the global drift angle exceeded 0.01 rad. The maximum base shear forces are close in the Y direction of RC specimen and the X and Y directions of PT specimen. The unloading branches of PT specimen show a trend of drift recovery. The time histories of base shear force show the elongation of the first mode period due to degrading of the slope. The natural period of RC specimen estimated by the White-nose input after the JMA-Kobe-100% test was 0.99 sec in X direction and 0.88 sec in Y direction, and that of PT specimen was 0.69 sec in X direction and 0.52 sec in Y direction.

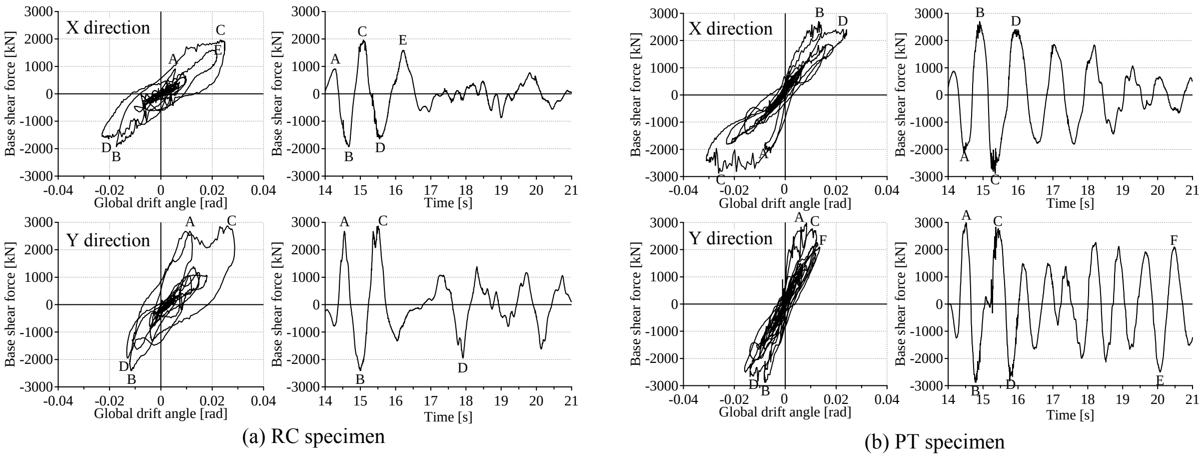


Figure. 14 Base shear force-global drift relationship and base shear force time history in JMA-Kobe-100% test

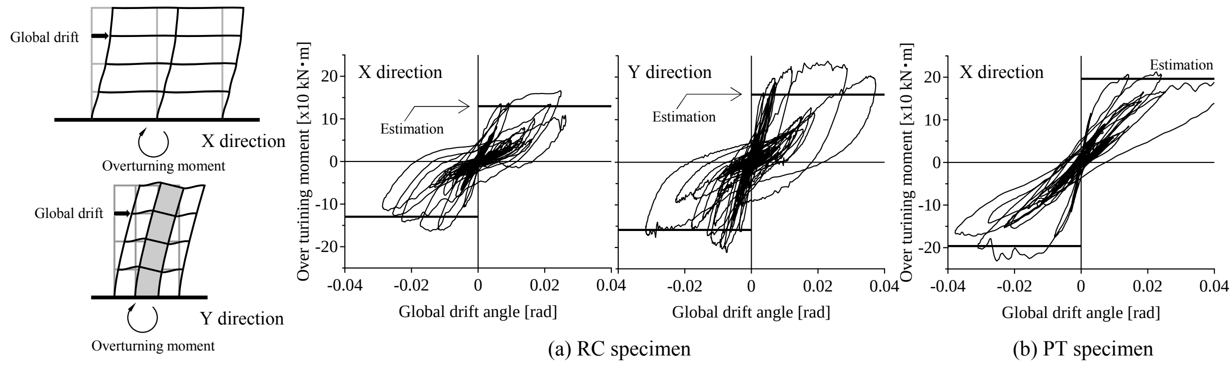
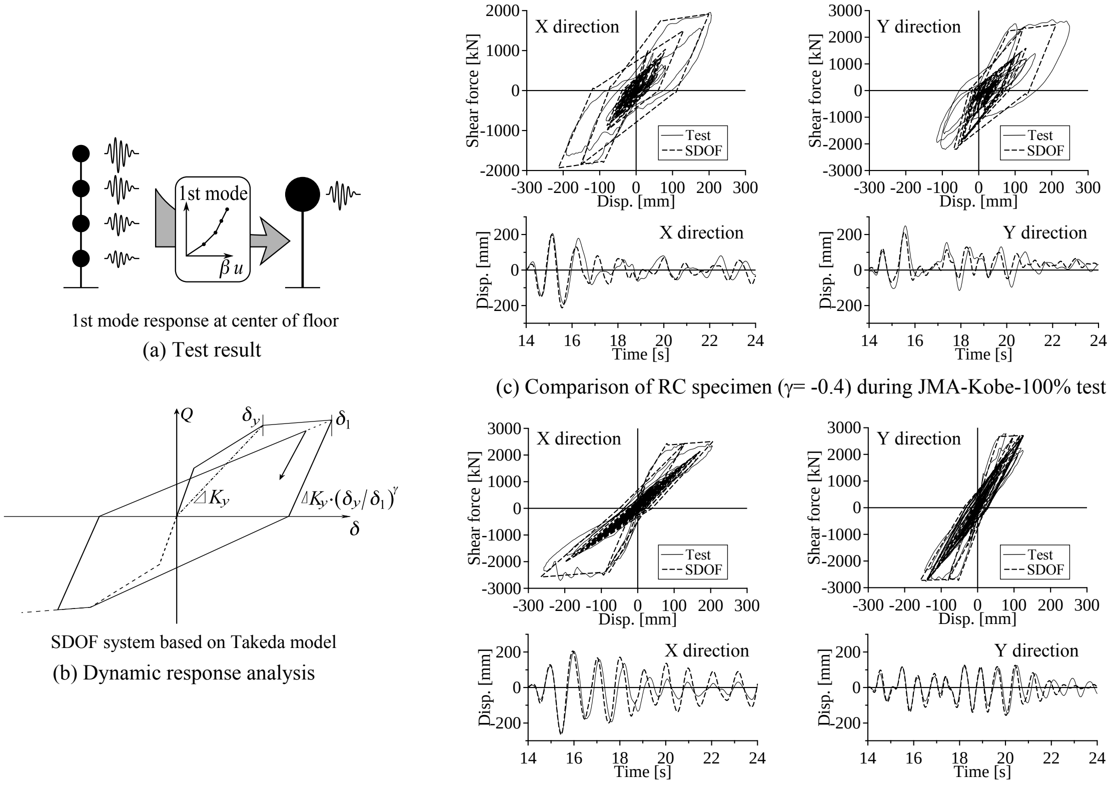


Figure. 15 Estimation of strength capacity based on Japanese Building Standard

For the X and Y directions of RC specimen and the X direction of PT specimen, seismic designs were applied according to the present Japanese seismic design code. Their capacity strengths were checked by using pushover analysis in reference to provisions of the document published by the Ministry of Land, infrastructure, Transport, and Tourism (2007, 2009). The flexural strengths of the beam, column and wall were evaluated reflecting the compressive strength of concrete and the yield strength of steel reinforcement given by the material test. Reinforcement of the top slab was reflected in the flexural strength of a beam section in negative bending (top in tension). Since the overturning moment at the base of the first story is mostly produced by the first mode response of a structure and is relatively insensitive in pushover analysis to the distribution of the lateral loads, experimental and pushover strength capacities can best be compared through the overturning moment. Figure 15 shows the relationship between overturning moment and global drift angle given from the JMA-Kobe-25% to JR-Takatori-60% tests, and the strength capacity estimated by the pushover analyses at the maximum inter-story drift angle of 0.02 rad is compared to it. In X direction, the strength ratio of the maximum test strength to the estimation is 1.3 for RC specimen and 1.1 for PT specimen, although the maximum test strength of PT specimen is 1.4 times that of RC specimen. In Y direction, the strength ratio for RC specimen is 1.5. This indicates that the presence of multi-story wall increases the actual difference.

Figure 16 shows the first mode responses of RC and PT specimens comparing to outputs of SDOF dynamic response analyses. The adopted SDOF system has a fitted skeleton curve and a tuned hysteretic rule defined by Takeda model. In the analysis, the test motions recorded in each directions were sequentially input from JMA-Kobe 25 % to JMA-Kobe 100 % with a long interval, and the initial damping ratio of 5 % was changed in proportion to the instantaneous stiffness. For both RC and PT specimens, the analyses well reproduced the large displacement responses in each directions. The factor to define unloading slope γ became -0.9 for PT specimen corresponding to the recovery shape, while that for RC specimen was -0.4. Those comparisons are simple, but the reasonable correlation essentially indicates that the dynamic response analysis given a proper model provides a reasonable performance assessment for the RC and PT building structures.



(a) Test result

(b) Dynamic response analysis

(c) Comparison of RC specimen (γ= -0.4) during JMA-Kobe-100% test

(d) Comparison of PT specimen (γ= -0.9) during JMA-Kobe-100% test

Figure. 16 SDOF dynamic response analysis using fitted skeleton curve and tuned Takeda model

7. SUMMARY AND CONCLUSIONS

Two full-scale four story reinforced concrete structures were tested side by side on the E-Defense shaking table. One structure was of conventional moment frame and shear wall design (RC structure) and the other was of post-tensioned moment frame and shear wall design (PT structure). Both structures were subjected to several high-intensity base motions. The RC structure remained stable throughout the tests, thus satisfying a life-safety performance objective. However, the structure sustained severe damage mainly in the wall bases and beam-column joints. The PT structure sustained small deformations in the shear wall direction and heavier damage with large deformations in the frame direction. The strength capacity of the test was 1.1 to 1.5 times larger than that estimated from the Japanese seismic design code. Given a fitted skeleton curve and tuned Takeda model, SDOF dynamic response analyses well assessed the first-mode response time histories of the tests.

AKCNOWLEDGEMENT

The authors acknowledge the support of the Ministry of Education, Culture, Sports, Science & Technology (MEXT) to carry out the test presented in this paper.

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