STRUCTURAL PERFORMANCE OF UNBONDED POST-TENSIONED PRECAST CONCRETE WALLS IN SHAKING TABLE TESTS AND DESIGN IMPLICATIONS

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Abstract: In 2010, one full-scale 4-story precast building was tested on the E-Defense facilities. The lateral-load-resisting system of this building consisted of unbonded post-tensioned precast walls in one direction. The performance of the precast walls during the test is presented here. Results showed that the walls performed well and no significant damage was observed. However, one of the walls sustained larger deformations than the other. In addition, the walls sustained significant out-of-plane rotations. Stresses in strands remained in the elastic range all through the test. Implications of test results for design procedures were also discussed.

Keywords: unbonded post-tensioning, precast concrete, shaking table, seismic design, self-centering.

1. INTRODUCTION

In 2010, two full-scale four-story buildings were tested on the E-Defense shaking table at the same time: a reinforced concrete building and a precast concrete building. The lateral-load-resisting system of the precast building consisted of bonded post-tensioned moment-resisting frames in one direction and unbonded post-tensioned (UPT) precast walls in the other. The description of the performance of the UPT walls during the test is presented in this paper. More information of the building specimens and overall results can be found in other publications [1,2].

2. OVERVIEW OF THE 2010 E-DEFENSE SHAKING TABLE TEST

Figure 1 shows the geometry of the precast building and its walls. The walls of the building consisted of precast panels (2500mm length and 250mm thickness) that were horizontally jointed at each floor level. The panel height was 3000mm in every floor except for the top panel, which had 3450mm height to accommodate anchorage systems. At the base, the unbounded tendons and a set of mild strength steel reinforcement, for dissipating hysteretic energy, crossed the horizontal joint. In upper wall panels, however, only the tendons crossed the horizontal joints. Two layers of horizontal and vertical reinforcements (D13) were arranged in the panels. Confinement was provided at the sides of walls (arranged at 75mm vertical spacing) with high strength steel reinforcement. Two unbonded post-tensioning (PT) tendons were used in the walls. Each tendon consisted of 10 strands of 15.2mm diameter. The initial prestress force applied to the tendons was about 53% of the measured yielding force. The energy dissipating (ED) reinforcement consisted of two sets of four bars with 22mm diameter, Fig.1c), and they were intentionally debonded over a length of 1500mm.

Steel fiber reinforcement was added only to the first and second panel of the north wall. The measured compressive strength of concrete was around 83.2 and 85.2MPa for elements without and with steel fiber reinforcement, respectively. The measured yield and ultimate tensile force in the PT strands were 253 and 278kN, respectively. The ED reinforcement had measured yielding and ultimate strengths of 385 and 563MPa, respectively.

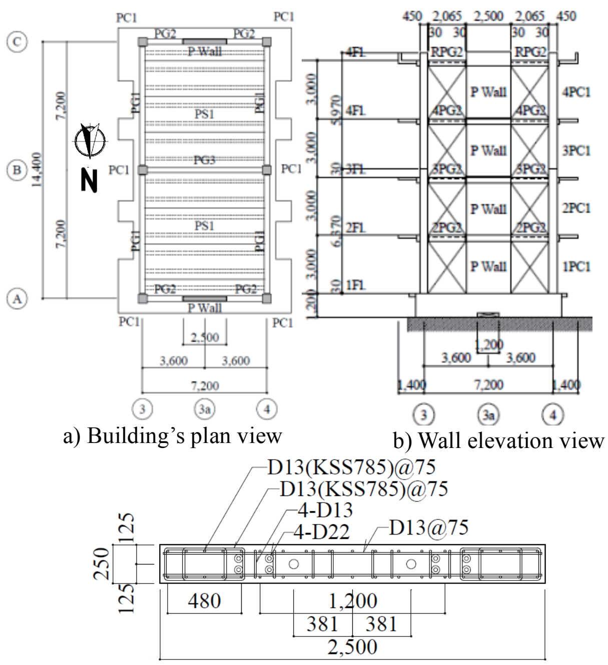
The building specimen was instrumented and monitored during the test program [1]. Fig. 2 shows the location of the transducers used in the base panel of the walls. The transducers S-N1, S-N2, S-S1, and S-S2 of Fig. 2 were only used in the south wall. In addition, load cells were placed at the top part of south wall to measure variation of forces in the tendon.

The JMA and Takatori ground motion records from Kobe earthquake in 1995 were used in the test. The North-South component of the JMA and Takatori records was applied in the wall direction of the building, whereas the East-West component of those records was applied in the moment-frame direction. The two horizontal and the vertical components of the records were applied simultaneously. The applied sequence of shaking motion was 10, 25, 50 and 100% of the JMA record; after that, 40 and 60% of the Takatori record was applied.

3. PERFORMANCE OF THE WALL SYSTEM DURING THE TEST

Herein, only results obtained from the JMA shaking motion are discussed. Fig. 3 shows the values of uplift at walls sides during the JMA-100% shaking motion. For the JMA-100% motion, the peak uplift in the south wall was about 40mm, which was 1.6 times larger than the peak uplift in the north wall. No noticeable damage was observed in the walls up to the JMA-50% shaking motion. However, partial spalling of cover concrete was observed in the south wall after the JMA-100% motion. Since steel fibers were used only in the north wall, a different degrading behavior of the two walls produced a torsional motion in the wall direction of the building [2].

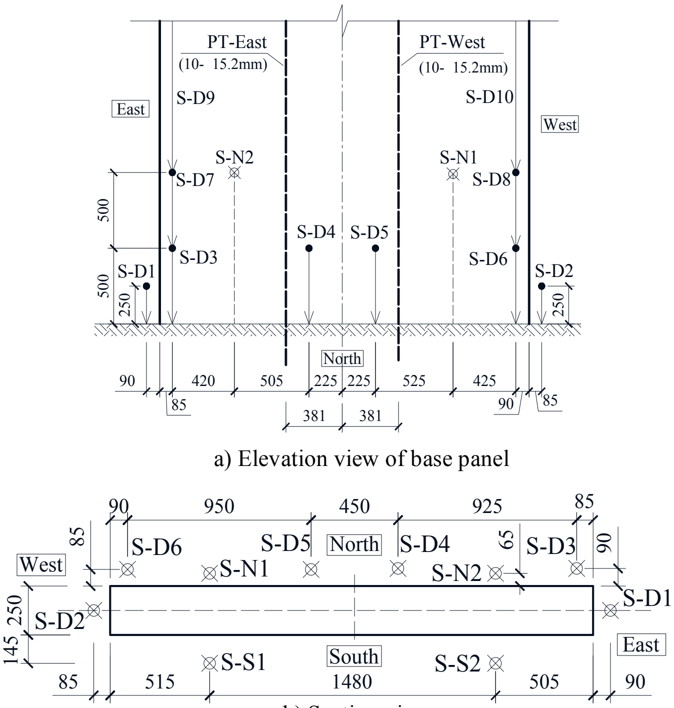
Figure 4 shows the time history of base rotation angles in the walls. Base rotation angles were estimated from displacement transducer measurements and by assuming a plane section at the base during gap opening. Positive rotation angles represent gap opening at the east side of the walls. Peak base rotation angles were larger in the south wall than the north wall for all the shaking motions used in the test. For the JMA-50% shaking motion, peak value of base rotation in the south wall was about 1.5 times larger than in the north wall. For the JMA-100% motion, the south wall had a peak rotation angle about 1.9 times larger than in the north wall.



a) Building's plan view　b) Wall elevation view

c) Wall reinforcement in base joint

Fig. 1 Details of precast building and configuration of reinforcement in walls [2](Units: mm)



a) Elevation view of base panel

b) Section view

Fig. 2 Details of instrumentation in precast walls. (Units: mm)

In addition to the in-plane rocking motion, out-of-plane rotations were estimated in the south wall by using readings of transducers S-S1, S-S2, S-N1 and S-N2 in Fig. 2. Fig. 5 shows out-of-plane base rotation angles for the JMA-50% and the JMA-100% shaking motions. Positive values in Fig. 5 indicate gap openings at the south side of walls. For medium levels of ground accelerations, the JMA-50%, the out-of-plane peak rotations were smaller than the in-plane ones. For the JMA-100% motion, however, the peak out-of-plane rotation was about 1.9 times larger than the peak in-plane rotation. It is important to notice that peak in-plane and out-of-plane rotations did not occur at the same time. Fig. 6 plots in-plane rotations against out-of-plane base rotations in the south wall. For the JMA-50% shaking motion it is seen in Fig. 6a) that in-plane rocking motion was dominant in the south wall. On the other hand, the bi-directional rotation became significant during the JMA-100% motion. Although the out-of-plane rotation did not produce observable damage in this test, influence of bi-directional rocking motion in precast walls needs to be further investigated. The bi-directional rocking motion may lead to compressive stress concentration in the corners of wall section and buckling of energy dissipating bars.

Figure 7a) shows the variation of gap opening (uplift) against base rotation angle and Fig. 7b) illustrates the variation of compressive strains at the sides of the walls against base rotation angle. For the JMA-100% motion, the peak compressive strain was about 3.1% and 1.2% for the south and the north walls, respectively. This difference represents almost 2.7 times larger strains in the south wall. The peak strain in the south wall (about 3.1% for the JMA-100% motion) is by far larger than the commonly accepted ultimate strain for unconfined concrete of around 0.4%.

Then, it can be argued that the confinement details played an important role in ensuring the integrity of wall corners.

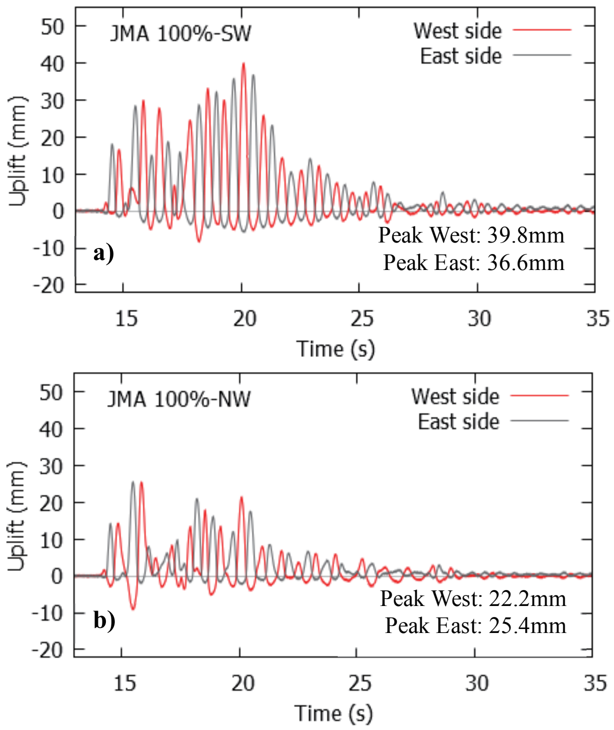


Fig. 3 Time history of uplift at base of a) South wall and b) North wall.

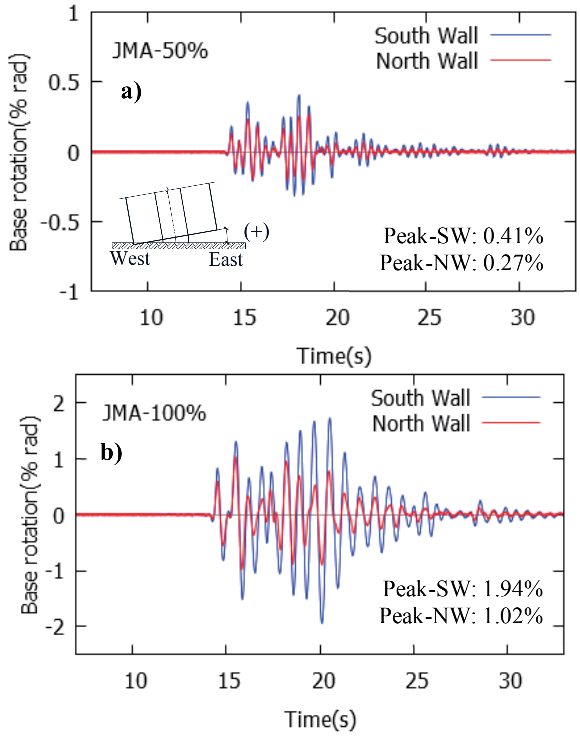


Fig. 4 Time history of estimated base rotation in the walls.

Figure 8a) shows the measured stresses (normalized by the measured yielding strength) in PT strands of the south wall. It can be seen that stresses in the strands remained below the measured yielding strength (f_{py}=1821MPa) even during the JMA-100% motion. The peak stress was 0.70f_{py} (1275MPa) and the initial stress introduced to the strands was about 0.53f_{py}. Fig. 8b) shows the measured stress in strands against base rotation angle.

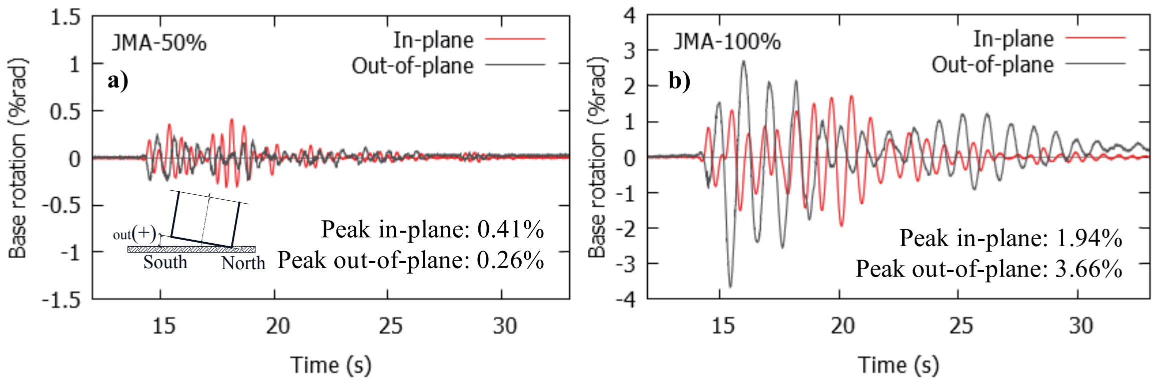


Fig. 5 Time history of estimated out-of-plane base rotation in south wall.

A set of strain gauges were attached in the debonded part of one ED bar (the easternmost bar in the south face) at the base of the south wall. Figs. 9a) and 9b) show the variation of strains against base rotation angle. Peak measured strains were 0.23 and 2.07% for the JMA-50% and JMA-100% motions, respectively. Until the JMA-50% shaking motion, the ED bar sustained a few strain cycles with inelastic strains (measured yielding strain ε_{sy} in the ED bar was 0.19%). Moreover, results revealed that the bar sustained some incursion into compressive strains at this shaking level. For the JMA-100% motion, the ED bar sustained several cycles of strains larger than ε_{sy}.

Finally, the summary of the response of the walls during the test is presented in Table 1. For all performance parameters and all the levels of shaking shown in Table 1, the south wall sustained larger peak values than the north wall. This different performance in the walls is mainly attributed to the torsional motion in the building due to differences in the degrading behavior of the walls (the north wall was steel fiber reinforced). It is also observed that peak in-plane base rotations were as large as corresponding peak drifts in the walls for the higher intensity shaking motion (JMA-100%). This indicates that all deformation was concentrated at the wall base because of the rocking motion. On the other hand, in-plane rotations made up around 50 and 70% of the drift at the roof for the JMA-25% and JMA-50% motions, respectively.

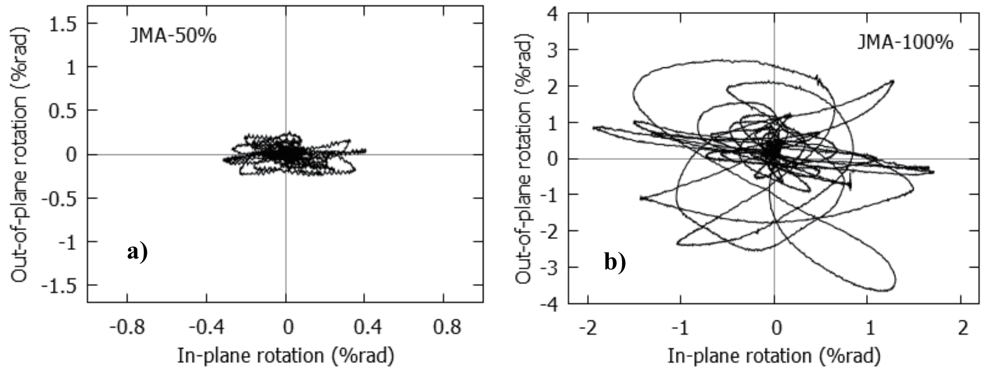


Fig. 6 In-plane against out-of-plane base rotation angles in the south wall.

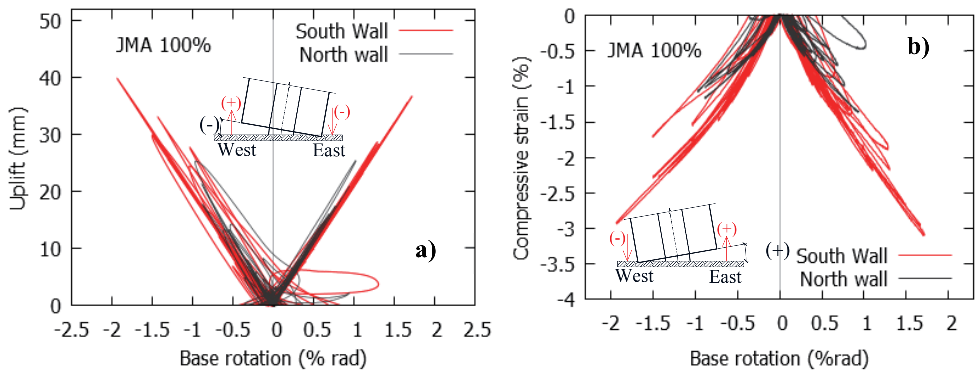


Fig. 7 In-plane base rotation against gap opening and compressive strains at wall ends.

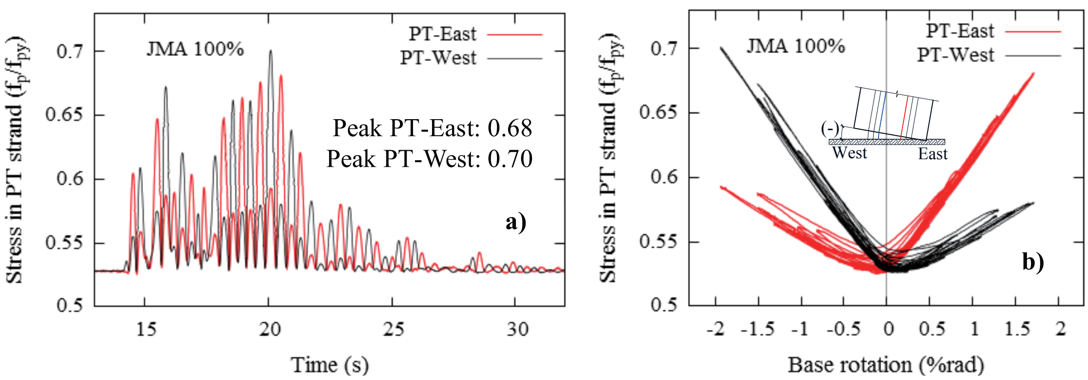


Fig. 8 Stresses in PT strands of south wall during JMA-100%.

4. IMPLICATIONS OF TEST RESULTS ON THE DESIGN OF UPT WALLS

Herein implications of the test results on the designing of UPT precast walls are discussed. The precast walls were designed according to ACI ITG-5.2 guidelines [3]. For design purposes, the UPT precast walls are considered to rock around their base joints as a rigid body and horizontal drift at the top of walls can be derived from the base rotation angle [3]. Moreover, the ACI ITG-5.2 document requires that the lateral drift of UPT precast walls during extreme lateral loads should be less than $0.70\Delta_{lim}$ [4], where Δ_{lim} is given as follows

$$0.90\% \le \Delta_{lim} = 0.8\%(h_w/L_w)+0.5\% \le 3.0\% \quad (1)$$

Equation (1) represents the maximum allowable drift in walls subjected to severe earthquakes, and it was proposed to avoid strength degradation [4]. For the walls previously discussed, the aspect ratio (h_w/L_w) was 4.98, which gives Δ_{lim} =4.5%. Then, drift limit should be Δ_{lim} =3.0%, according to Eq. (1). The peak roof drifts during the JMA-100% motion were 1.98 and 1.11% for the south wall and the north wall, respectively. These values are lower than $0.70\Delta_{lim}$ =2.1%. Thus, the precast walls satisfied the drift limitation requirement without any observed strength degradation during the test.

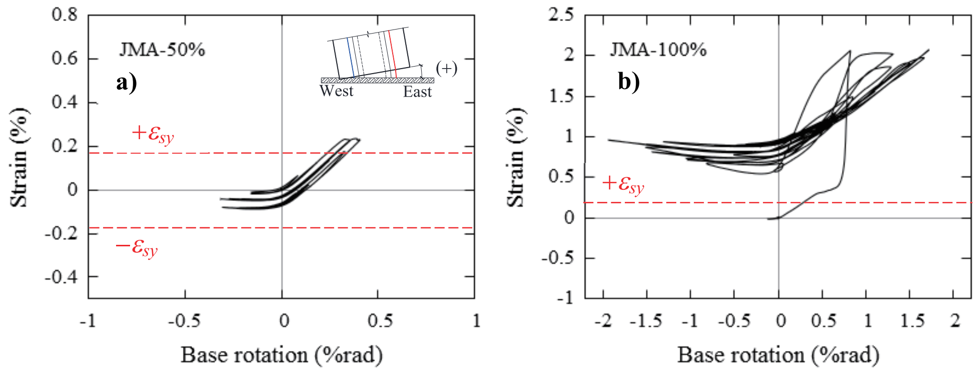


Fig. 9 Values of strains in one ED bar of south wall.

Table 1 Peak response values in the UPT precast walls.

Peak values	JMA-25%		JMA-50%		JMA-100%	
	SW	NW	SW	NW	SW	NW
Drift at roof (%rad)	0.21	0.14	0.61	0.38	1.98	1.11
In-plane base rotation (%rad)	0.10	0.07	0.41	0.27	1.94	1.02
Out-of-plane base rotation (%rad)	0.10	-	0.26	-	3.66	-
Uplift - West side (mm)	1.11	0.57	6.66	4.25	39.79	22.18
Uplift - East side (mm)	2.12	1.50	8.90	5.81	36.60	25.39
Compressive strain - West side (%)	0.13	0.09	0.43	0.28	2.93	1.17
Compressive strain - East side (%)	0.19	0.13	0.64	0.32	3.13	1.08
Stress in west tendon (f_p/f_{py})	0.53	-	0.56	-	0.70	-
Stress in east tendon (f_p/f_{py})	0.53	-	0.57	-	0.68	-
Strain in ED2-east rebar (%)	0.08	-	0.23	-	2.07	-
Strain in confinement steel (%)	0.004	-	0.011	-	0.024	-

According to ACI ITG-5.2, the stresses in PT strands should be smaller than the yielding strength under the maximum seismic displacement demand, which is $0.70\Delta_{lim}$, to maintain the self-centering capacity in the walls [3]. As it was noted before, the maximum stress in PT strands during the test was about $0.70f_{py}$. Moreover, the residual deformation (base rotation angle) after the JMA-100% motion was relatively small, as it can be seen in Fig. 4b). Then, this provision seems adequate to ensure the self-centering capacity of the system during large earthquake loads.

Repeating cyclic loading under high stresses in PT strand-anchorage systems may lead to premature failure and undesirable performance of UPT precast elements [5]. The appendix B of the New Zealand Standard for Concrete Structures, (NZS3101:2006), indicates that anchorage systems should be able to sustain more than 50 cycles at load levels of 50 to 80% of the nominal tensile strength of strands [6]. The International Code Council Evaluation Service (ICC-ES) makes a similar requirement for anchorages, indicating that cyclic loading levels should be 40 to 80% of the nominal strength and with a frequency of 1 to 3 Hz [7]. On the other hand, the Architectural Institute of Japan (AIJ) recommends that anchorage for unbonded strands should resist more than 200 loading cycles at load levels of 50 to 90% of the nominal strength [8]. For the walls in the test, the measured stresses ranged from 50% (initial PT force) to

67% of the nominal strength (1882MPa) for the strong shaking level. Additionally, the number of loading cycles was about 26 in the main part of the JMA-100% shaking motion, which represents an average frequency of 2.1Hz.

The ACI ITG-5.2 also requires that strains in energy dissipating bars should be less than 85% of its ultimate strain capacity (ε_{su}) during large earthquake levels [3]. From the tensile test on some bar samples used as energy dissipating reinforcement, the ultimate strain was about ε_{su}=20%. The maximum measured strain in the ED bar of the south wall was about 2.07%, for the JMA-100% shaking motion, which was lower than 0.85ε_{su}=17%. Then, it is reported that the ED bars in the walls were able to provide energy dissipation without any apparent fracture.

5. CONCLUSIONS

The precast walls performed satisfactorily and no significant damage was observed in the test. The south wall was subjected to larger deformations than the north wall due to the torsional response of the building. In addition, the influence of steel fibers in the panels proved to be important in controlling concrete spalling of the north wall.

Significant out-of-plane base rotation (about 3.7%) was observed in the south wall during the JMA-100% motion. Although the out-of-plane rotation did not cause observable damage in this test, influence of bi-directional rocking motion needs to be further investigated.

Large compressive strains occurred in the wall toes during the JMA-100% motion, reaching a maximum compressive strain of 3.13% in the south wall. Despite this level of compressive strain, the walls suffered no damage in the core concrete. However, some cover spalling was observed in the south wall during this test. Thus, appropriate confinement reinforcement in the wall section is essential to avoid degradation of load carrying capacity.

The stresses in PT strands of the south wall remained in the elastic range and no fracture was observed. The maximum measured stress in strands was about 70% of its measured yielding strength and the walls sustained slight residual deformations. Then, the walls kept their full self-centering capacity all through the test.

Measured strains in one of the energy dissipating bars on the south wall showed that it underwent several cycles of inelastic strain. The maximum measured strain was about 2.07%, which was smaller than the limit strain (about 0.85ε_{su}=17%) according to the ACI ITG-5.2 document.

The test results indicated that requirements in the ACI ITG-5.2 seemed to be adequate to avoid premature failures of the UPT precast walls under severe earthquakes. This test, however, raised some important issues not clearly defined in the design guidelines such as out-of-plane rocking motion and influence of fiber content in concrete panels.

ACKNOWLEDGMENTS

Data of the test reported here were provided by the National Research Institute for Earth Science and Disaster Resilience (NIED) (https://www.edgrid.jp/data). Financial support of the Peruvian National Council of Science, Technology and Technological Innovation (CONCYTEC/CIENCIACTIVA) for the doctoral studies of the first author is duly acknowledged.

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