Seismic Behavior of Welded Precast Panel Connections

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ABSTRACT

The behavior of connection between precast panels is important as it controls the failure mode of the shear wall. Therefore, it is aimed to test the behavior of different type of precast panel connections in this study. One monolithic reference model and two precast panel models having different horizontal connection details were produced. The behavior of connections was tested under reversed-cyclic lateral loads. It was concluded that welded connections can be designed in high and normal ductility levels.

Keywords: Seismic Strengthening, External Shear Wall, Precast Panel Connection

INTRODUCTION

Strengthening of the structures with deficient seismic performance is a significant subject for many countries. Numerous methods were developed to strengthen existing buildings. RC infill walls, jacketing, and steel shear walls are some of them. These methods are both introduces some difficulties for construction and need building’s to be out of service. Besides, these are also time-consuming and expensive [1]. The use of external shear walls for strengthening of existing buildings was developed to overcome these difficulties. The studies on external shear wall applications in parallel to facades [2] and those in vertical to facades [3, 4] are available in literature. In these studies, external shear walls were produced as cast-in-place and connection to the existing structure was provided by adhesive anchors. In multi-storey structures, there are several difficulties in the application of cast-in-place construction of external shear walls.

On the other hand, the strengthening made with precast members provides ease of application. Several studies were conducted for the use of precast members for the strengthening of buildings [5-9]. Although, the application of precast panels from outside of the buildings provides a considerable ease of application, the production of panels in multi-piece becomes a necessity with increasing heights of buildings. In the literature, there are several studies in the subject of horizontal panel connections of Precast RC Panel walls [10-12]. Besides these studies; in previous earthquakes, it was seen that prefabricated panel systems show good performance [13-17].

In this study, external shear walls were produced as precast panels. Panel connections were designed for one-face only to simulate the real case. In this context, two precast panel specimens having different horizontal connection details and one reference specimen without any connections were prepared. The models were tested under reversed-cyclic lateral loads.

EXPERIMENTAL STUDY

Specimens

This study consists of three models. The reference model (RM) and panel models with two different connection types were constructed in one-third scale. Labeling and description of models are given in Table 1. Dimensions and detailing of the specimens are given in Figure 1.

Table 1 Experimental Models

Model Name	Panel-Panel Connection Type	Description
RM	Without connection- one casting	Reference model
WHC1	Models with welding and conjunction	U-cross sectioned steel components which are dry in horizontal with clamp and steel sheet supported conjunction
WHC2		Conjunction with U-cross sectioned, steel, dry in horizontal components

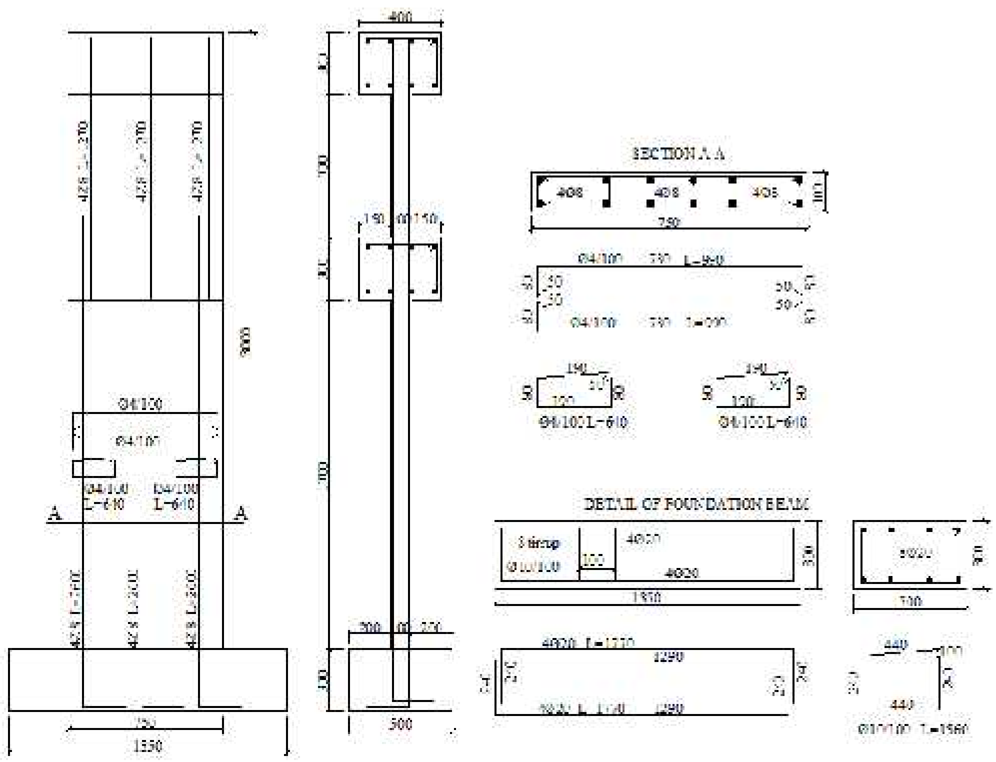


Figure 1 Reinforcement details of the reference model

Panels were produced in two pieces for WHC1 (Welded Connection in Horizontal) model. At the panel ends, where connections are to be realized, cold formed U sections having a thickness of 5 mm which are made of A36 steel. Longitudinal reinforcements of the shear walls were welded to the internal surfaces of these steel end section (Figure 2). During the assembly, these end sections were welded with fillet weld. Furthermore, steel plate (A36) having dimensions of 100.200 was welded to both end sections. For all welding works, weld thickness was 3.0 mm. All connections were formed with electric arc welding by alkali electrodes.

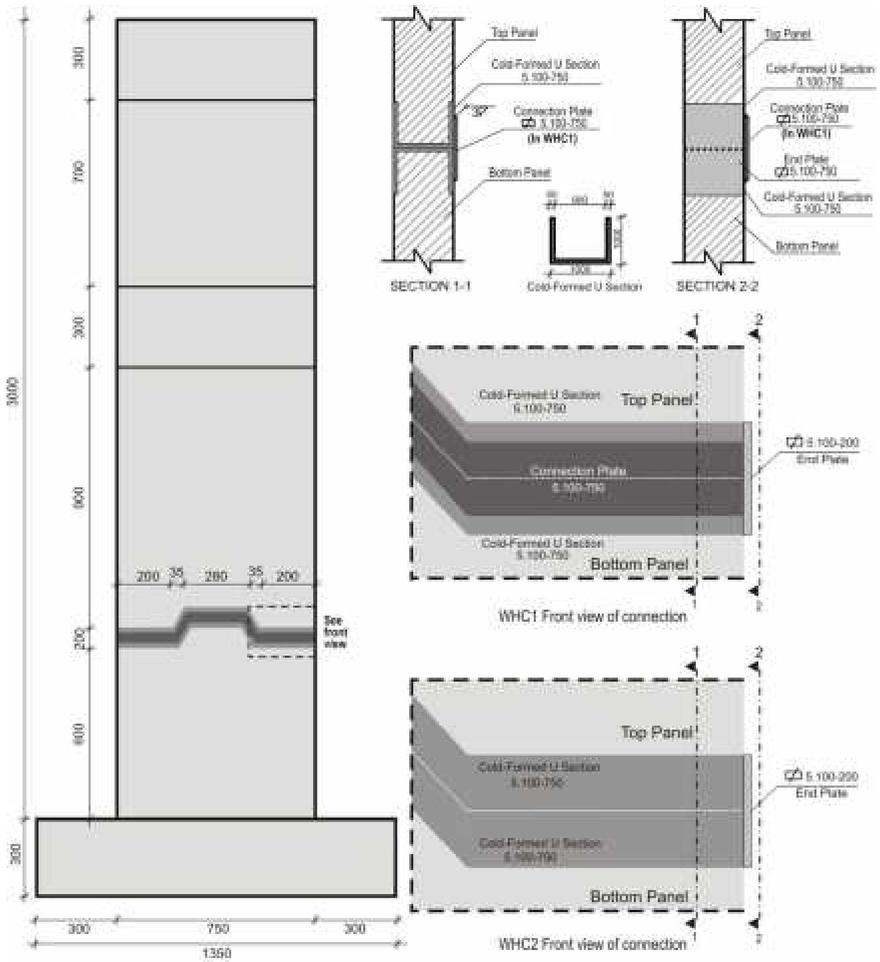


Figure 2 Schematic view of WHC1 and WHC2 models and connection details.

WHC2 (Welded Connection in Horizontal) model has the same properties as WHC1 model except that it has no connection plates, welded over U-end sections. Connection was provided by welding U-end sections from one side (Figure 2).

Materials

In the production of specimens, ready-mix concrete with maximum grain size of 16 mm was used. Mean concrete compressive strength of each model is given in Table 2 for 28th day and the day of experiment.

Table 2 Mean concrete compressive strength of test models

Experiment element	fc (MPa) (Mean compressive strength)

	28 Days	Experiment Day
RM	42.42	50.23
WHC1	42.42	50.23
WHC2	36.09	41.81

Moment and Shear Capacity of Connections

In this study welded connections were used. In WHC1 and WHC2 models, connection of panels was provided by welding. Shear and moment capacity of connections and the shear wall are calculated without considering material safety factors. Those capacities are compared with design forces in Table 3. When the panel reaches its moment capacity at the bottom end, the design moment (M_{dc}) at the connection level, which is located at 700 mm height should be smaller due to linear moment distribution throught the wall height. That reduction is carried out for the calculation of M_{dc}. It is seen that connections are considerably safe against shear forces. However, critical effects which define the safety of connections are flexural forces.

Table 3 The sufficiency of sectioning force and moment capacities of connection regions

Specimen	Moment capacity of the wall M_r (kNm)	Design moment for the connection M_{dc} (kNm)	Design shear force for the connection V_{dc} (kN)	Moment capacity of the connection M_{rc} (kNm)	Shear capacity of the connection V_{rc} (kN)
RM	116.19	No connection	40.77	No connection	
WHC1		87.65		141.59	809.13
WHC2				91.03	404.56

Test Setup

Experimental studies were conducted in Pamukkale University Earthquake and Structural Technologies Research Laboratory. Within this study, specimens were tested under reversed-cyclic loading applied from top of the panel walls. Overview of loading system is given in Figure 3. Sliding supports were used at the top of panels to prevent the out of plane-behavior of specimens (Figure 3).

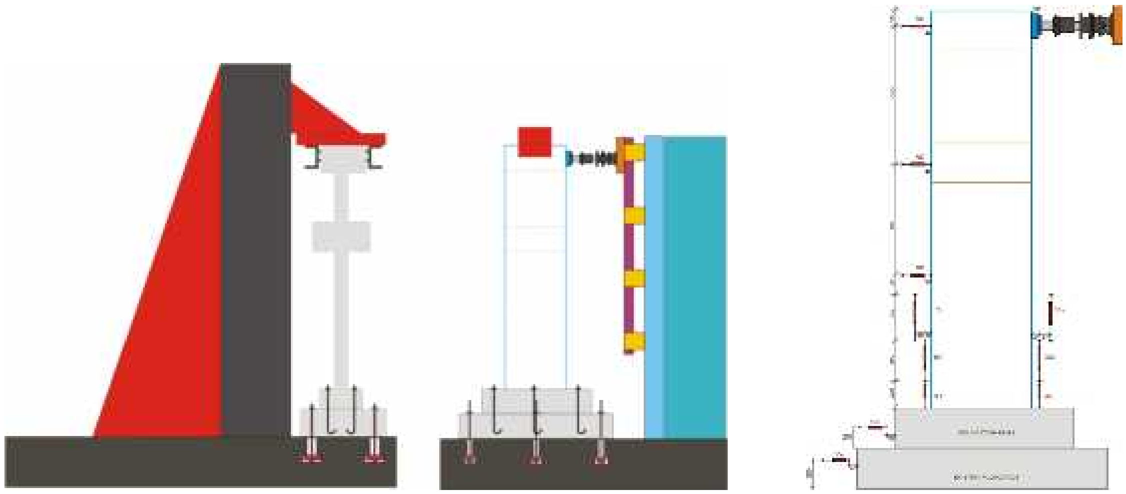


Figure 3 Overview of test setup and placement of data acquisition

Displacement transducers were placed to the same places for each model (Figure 3). Curvature measurements were performed for three different regions. The average curvature measurements were carried out in the region with lengths of 200 mm, 300 mm, and 350 mm from the bottom. The third region where the last measurement was done also covers the connection.

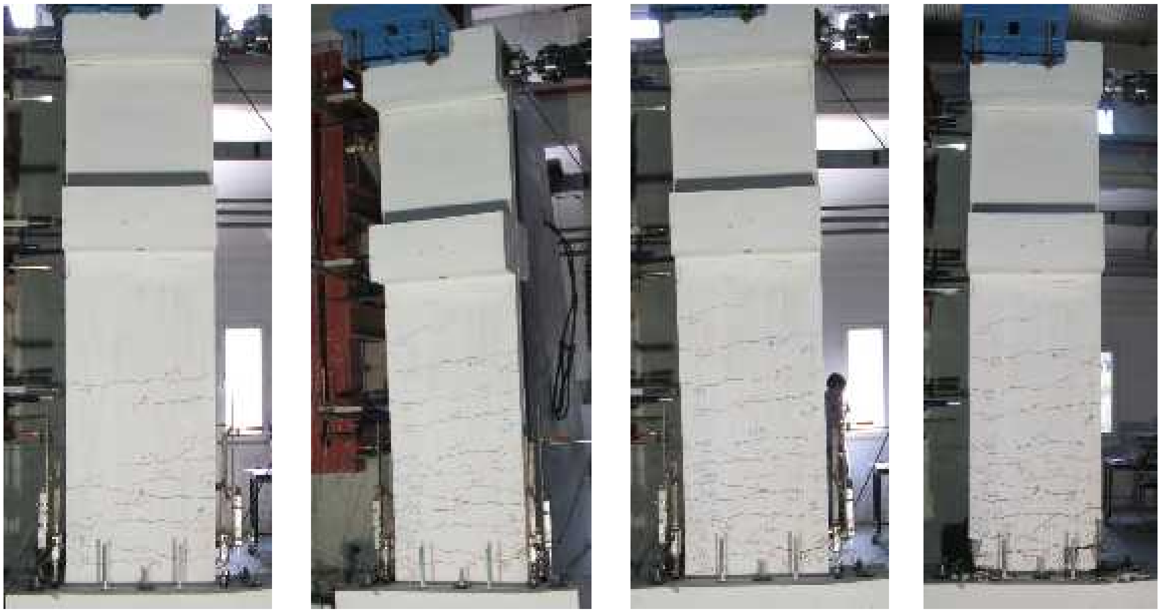
Lateral movement and rotation of base were prevented. Some displacement transducers were used to measure whether there has been any sliding in the interface between panel and rigid base. In RM model in the experimental study and in all other experiment models, it was seen that there is not a sliding formation in the elements which provide connection in panel base. Load measurements in the experimental study were done with the load cell placed between piston and panel. Reversed cyclic loading was applied to the specimens.

TEST RESULTS

All experiments within this study were performed as displacement controlled. Experiment cycles were started with +5 mm and continued up to +/- 100 mm peak displacement depending on the stability of specimens. The deformations and damages on specimens were inspected after each step.

Reference Model (RM)

RM was tested under reversed-cyclic loading. Damage conditions of panel at the moments of 0.5 %, 1 %, 2 %, and 3.3 % lateral sway were given in Figure 4.



0.5% 1.0% 2.0% 3.3%

Figure 4 Observed damages at different sway levels for RM

As a result of the experiment, bending and shear cracks were observed in the first 180 cm- and mostly in first 60 cm- height of the panel wall. RM element exhibited a ductile behavior and no loss in load-bearing capacity is observed up to + 99 mm peak displacement. After the experiment, maximum base shear force and moments are found to be 43.26 kN and 118.96 kNm respectively. Moment-curvature hysteresis for $1^{st,}$ 2^{nd}, and 3^{rd} regions and force-displacement cycles are given in Figure 5. It was seen that nonlinear behavior mostly formed at the first region.

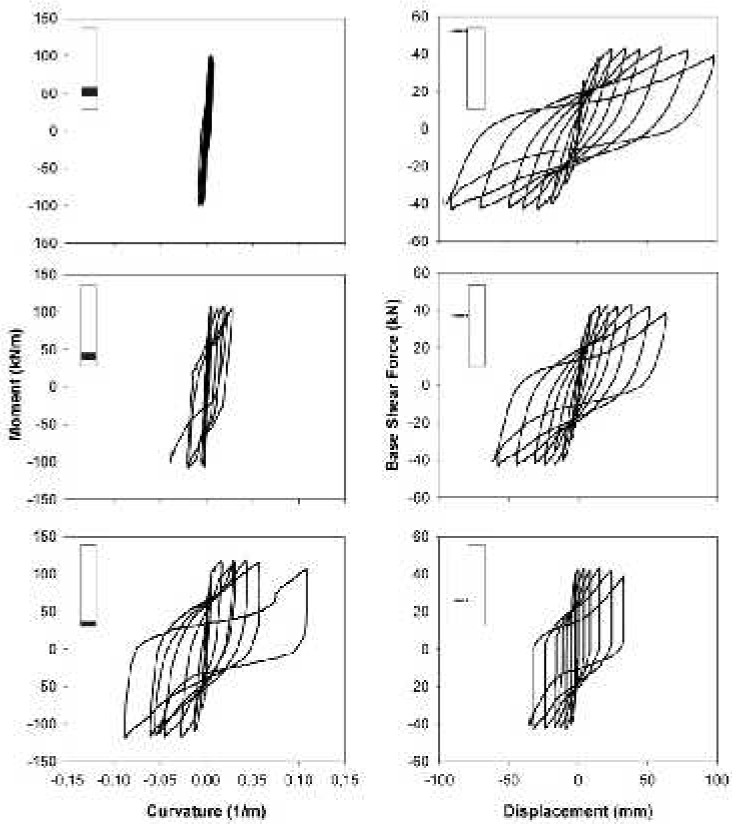
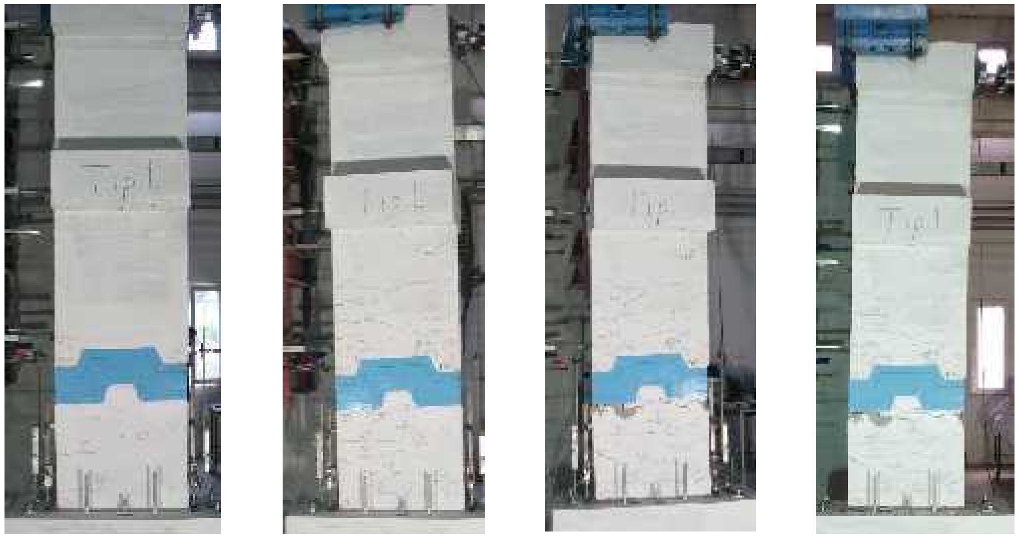


Figure 5. Moment-Curvature and Force-Displacement hysteresis for RM

WHC1 Model

In WHC1 model, experiment started with +5 mm and ended with –70 mm peak displacement. A clear capacity loss started at the displacement level of 42.03 mm in the push direction of WHC1 model and maximum base shear force and moment were found to be 39.25 kN and 107.94 kNm respectively. After the experiment, the damages of panel at the sway of 0.5 %, 1 %, 2 %, and 3.3 % are given in Figure 6.



0.5% 1.0% 2.0% 3.3%

Figure 6 Observed damages at different sway levels for WHC1

No damage formation was observed in the welds connecting WHC1 panels. Although flexural cracks were widely seen throughout the panel, at –70 mm peak displacement, panel reached to the collapse after crushes around the connection and failure in reinforcements.

Moment-curvature hysteresis for $1^{st,}$ 2^{nd}, and 3^{rd} regions and force-displacement cycles are given in Figure 7. In first measurement region, some plastic deformations were observed and panel reached its flexural capacity. However, in contrast to RM, no plastic deformations were occurred in second region, where curvature has changed linearly. When the panel reached 40 mm top displacement, connection failed and it was subjected to a significant capacity loss. Due to this damage, a rapid curvature increase was observed while moment decreases in the connection region. Due to the increased top displacement because of damage occurred in the connection, the demand for curvature in the panel base has decreased. Final curvature values were measured as 0.04 rad/m and 0.01 rad/m for the first and second regions respectively; while that in the connection region increased to 0.02 rad/m. Although, nonlinear curvature is not observed in this region for RM, curvature increase arose with significant capacity loss in case of welded-connection. Flexural behavior at the connection was considerably brittle. However, this brittle behavior happened due to break of connection between reinforcement bars and the end plate. It is clear in Figure 6 that there is no damage in welds between panel connection plates.

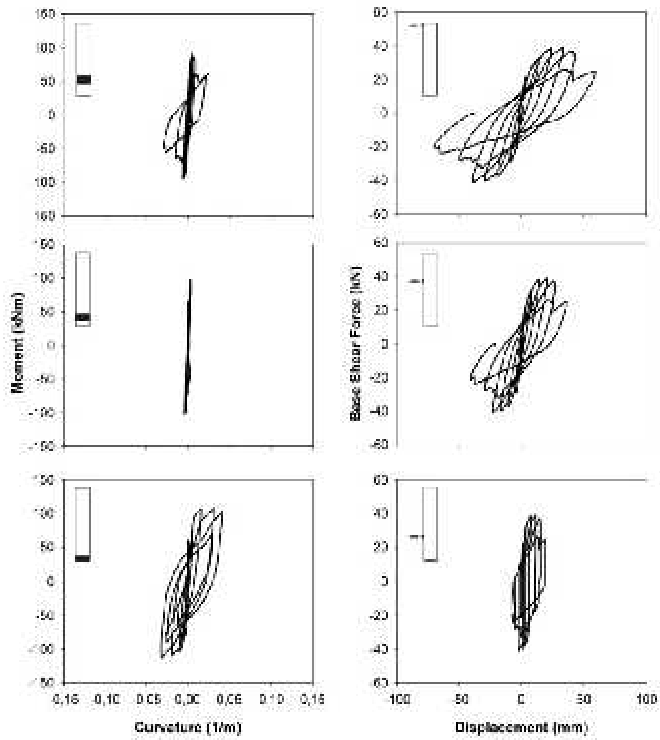
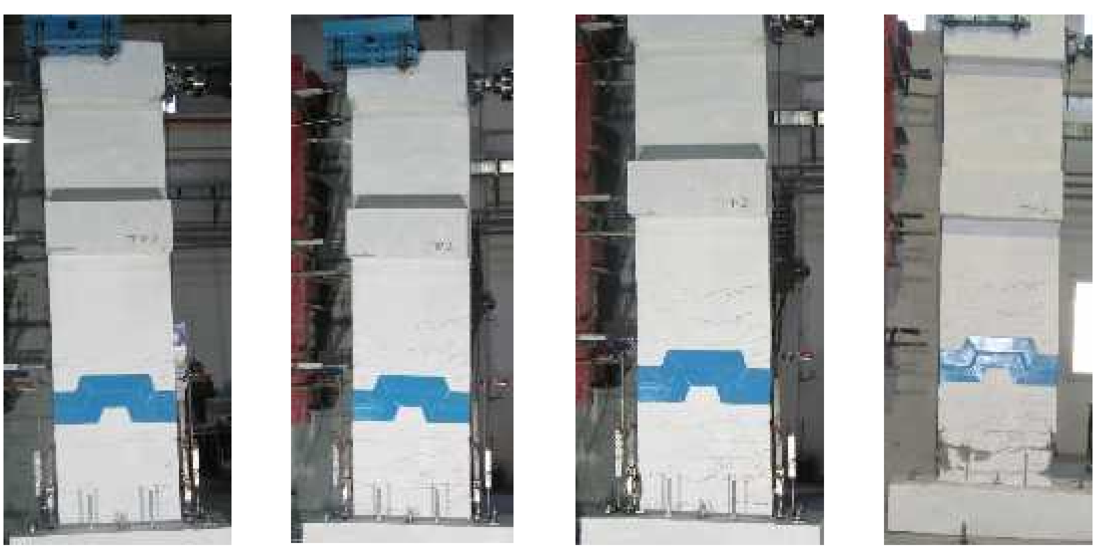


Figure 7 “Moment- Curvature” and “Force-Displacement” curves for WHC1

WHC2 Model

For this specimen, similar damage pattern was observed with RM. No damage was observed in the welds which connect panels. WHC2 model shows a ductile behavior like Reference Model. A clear capacity loss was formed at 80.92 mm top displacement in the pull direction. Maximum base shear force and maximum moment were found to be 39.25 kN and 106.32 kNm respectively. The damage schemas at the sway levels of 0.5 %, 1 %, 2 %, and 3.3 % are given in Figure 8.



0.5% 1.0% 2.0% 3.3%

Figure 8. Observed damages at different sway levels for WHC2

Moment-curvature hysteresis for 1st, 2nd, and 3rd regions and force-displacement cycles are given in Figure 9. Nonlinear behavior was not observed in the connection region. Curvatures in the 1st and 2nd region still continued to increase after yielding of longitudinal reinforcements. Plastic curvatures were observed in the first 500 mm of panel height. Nonlinear behavior occurred throughout this height and panel reached the collapse mode with rebar failure in the panel base and concrete crushes. Due to well-performance of the connection, panel exhibited a ductile behavior (Figure 9).

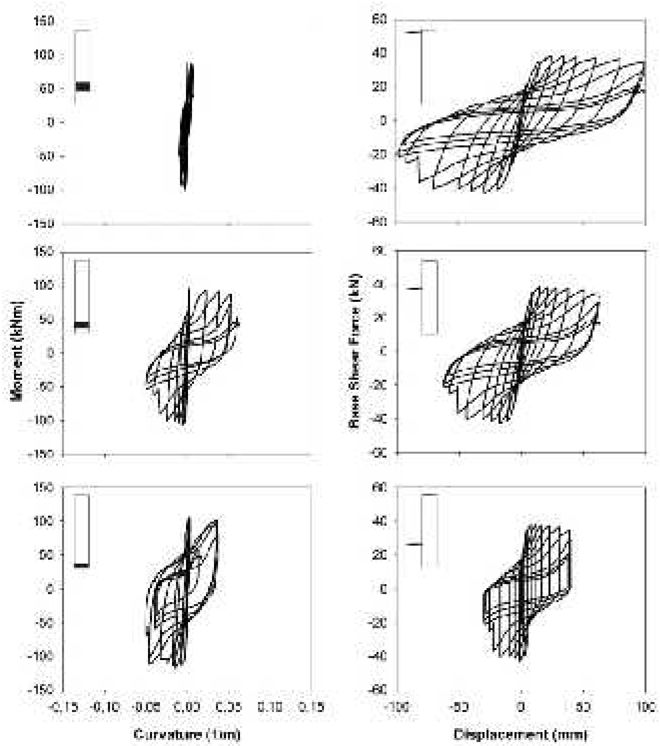


Figure 9. "Moment- Curvature" and "Force-Displacement" curves for WHC2

CONCLUSIONS

In scope of this study, it is aimed to investigate the behavior of precast panels connected to form a strengthening shear wall. A reference monolithic model and two panel models were tested under reversed-cyclic loads.

Reference model behaved in a ductile manner and a plastic hinge is formed at the bottom of the wall. The plastic damages were concentrated mostly in the 200 mm height of the model. It has a displacement ductility of 7.07.

Precast panels were connected with welded connections to form a similar geometry with the reference model. WHC1 model failed under cyclic loading. WHC2 has a similar ductility to that of the reference specimen. Although, WHC1 suffered from the connections, it could reach the flexural capacity at the bottom. Some damages were experienced at the bottom of this model; however its ductility was not as high as reference model and WHC2.

In design stage, shear and flexural capacity of connections were both checked. Flexure was the dominating criteria for the design of the connections. It is concluded that precast strengthening panels can be connected with welded end plates. However, the designer should be aware of the ductility of the connections. Welds between end plates and reinforcements as well as welds between connection plates should satisfy enough shear and flexural capacity and assure high quality of construction.

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REFERENCES

[118] Kaplan, H. and Yılmaz S. (2012) Seismic strengthening of Reinforced concrete buildings. *Earthquake-Resistant Structures - Design, Assessment and Rehabilitation.* Chapter 16, ISBN 979-953-307-680-4.

[119] Kaplan, H. Yılmaz, S. Çetinkaya, N. and Atımtay E. (2011) Seismic strengthening of RC structures with exterior shear walls. *Indian Academy of Sciences*.,36(1), 17–34.

[120] Kaplan, H. Nohutcu, H. Çetinkaya,N. Yılmaz, S. Gönen, H. and Atımtay E. (2009) Seismic strengthening of pin-connected precast concrete structures with external shear walls and diaphragms. *PCI J,* 54(1), 88-99.

[121] Kaltakçı, M. Y. Öztürk, M. and Arslan M. H. (2010) An experimental investigation for external RC shear Wall applications. *Nat. Hazards Earth Syst. Sci.:* 1941-50.

[122] Yüzügüllü Ö. (1979) Strengthening of reinforced concrete frames damaged by earthquake using precast panel elements. *TUBITAK Project* No.MAG-494.

[123] Frosch, RJ. Wanzhi, L. Jirsa JO. and Kreger ME. (1996) Retrofit of non-ductile moment-resisting frames using precast infill wall panels. *Earthquake Spectra* 12(4), 741-760.

[124] Baran, M. and Tankut, T. (2011) Experimental study on seismic strengthening of RC frames by precast concrete panels. *ACI Struct. J*, 108(2), 227-237.

[125] Yılmaz, S. Tama, YS. and Kaplan, H. (2010) Design and construction of external precast shear walls for seismic retrofit. *14th European Conference on Earthquake Engineering* Ohrid, Macedonia, 1238.

[126] Yılmaz, S. Çetinkaya, N. Nohutcu, H. Çalışkan, Ö. and Çırak, İF. (2010) Experimental Investigation of anchors in external strengthening shear walls. *TUBİTAK Project* No.107M572.

[127] Soudki, KA. Jeffrey, SW. Rizkalla, SH. and Bruce B. (1996) Horizontal connections for precast concrete sheare wall panels under cyclic shear loading. *PCI J* 41(3), 64-80.

[128] Soudki, KA. Rizkalla, SH. and LeBlanc B. (1995) Horizontal connections for precast concrete sheare walls subjected to cyclic deformations part 1: mild steel connections. *PCI J 40*(4), 78-96.

[129] Soudki, KA. Rizkalla, SH. and Diakiw RW. (1995) Horizontal connections for precast concrete sheare walls subjected to cyclic deformations part 2: prestressed connections. *PCI J* 40(5), 82-95.

[130] Polat, U. Göktepe, S. and Gürses E. (2000) Behavior and Seismic Rehabilitation of Prefabricated Frame and Panel Wall Structural Systems. *Proceedings of 10th Symposium on Prefabrication*, İstanbul, Turkey, 117-126.

[131] Sachanski, S. (1978) Earthquake resistance of precast structures. Proc. of the *6th European on earthquake engineering*. Dubrovnik, 3-28.

[132] Hashim, MSAW. and, Sinan, YHS. (1991) Prediction of Ultimate Shear Strength of Vertical Joints in Large Panel Structures. *ACI Structural Journal*, 88-S25.

[133] Booth, E. (1994) Concrete structures in earthquake regions, design and analysis. Longman Scientific & Tecnical, Harlow Essex, England.

[134] Ersoy, U. Özcebe, G. And Tankut, T. (2000) Damages observed in precast structures in 1999 Marmara and Düzce Earthquakes. *Earthquake and Prefabrication Journal* (In Turkish).