HORIZONTAL CONNECTIONS FOR PRECAST CONCRETE LOAD BEARING SHEAR WALLS IN SEISMIC ZONES

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ABSTRACT

The precast concrete shear wall system is becoming very popular in North America for low, medium, and highrise construction due to the economical advantages possible with the ease and speed of assembly, and the high quality of the precast panels. The connections between panels are extremely important since they affect both the speed of erection and the overall integrity of the structure. This paper presents the results of the last two phases of a six-year research program conducted to investigate the behaviour and the capacity of fifteen connection configurations used for precast load-bearing shear wall panels subjected to monotonic shear loading and large reversed cyclic loading conditions.

Connection configurations include dry-packed plain surface, dry-packed plain surface with continuity reinforcement, dry-packed with two types of mechanical shear connectors in addition to the continuity bars, dry-packed with post-tensioning, and two types of dry-packed multiple shear keys. The research also include connections which support hollow-core slab with and without post-tensioning. The performance of new innovative connections, such as partial or complete debonding of the reinforcement continuity and the use of energy dissipating mechanical connections for the continuity reinforcement, were also examined. Three different levels of load normal to the connection to simulate the effects of gravity load were used.

The research findings of this investigation were implemented in the design and construction of three 32-stories apartment buildings in Winnipeg, Manitoba. The connections considered in this study provided significant savings in construction time and cost.

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INTRODUCTION

At present, the precast concrete shear wall system is being designed and constructed in non-seismic zones. The use of such a system is very limited in seismically active regions due to the lack of knowledge of how this type of construction performs under seismic loading conditions. The structural integrity of the precast wall building is greatly influenced by the behaviour of the connections between the precast wall-elements. Current design codes do not specifically address the seismic design of the precast connections. The information available in the existing literature is not directly applicable to precast shear wall connections and does not consider the cyclic behaviour of such connections. Therefore, in order for precast concrete shear wall panel systems to gain acceptance and be a competitive construction system in seismic regions, the cyclic behaviour of the connections between the precast members must be studied.

This paper presents results of an extensive experimental program on the cyclic behaviour of horizontal connections for precast concrete shear wall panels subjected to large reversed cyclic inelastic deformations. This research is part of a multiphase experimental program at the University of Manitoba on the static and cyclic behaviour of precast wall connections. A total of forty-three prototype precast concrete shear wall panel specimens were used to test fifteen different connections including new innovative connections and configurations currently used by the construction industry. The results of earlier phases on the monotonic shear behaviour of typical precast shear wall connections were published by the PCI Journal (Foerster, et al., 1989, Serrette, et al., 1989, Hutchinson, et al., 1991). The objective of the current research is to study the behaviour of the connection under reversed cyclic pure shear loads (West et al., 1993) and combined cyclic flexure and shear (Soudki, et al., 1993). All specimens were subjected to a constant axial load normal to the connection to simulate gravity loads. The test results were used to determine stiffness, strength, ductility capacity, energy dissipation, modes of failure, and contribution of the different mechanisms to the overall behaviour. Based on the experimental results, design recommendations for precast connections in seismic zones, are presented.

EXPERIMENTAL PROGRAM

A total of seventeen specimens were tested in cyclic shear and combined cyclic shear and flexural loading. Descriptions of the specimens tested are given in Table 1. Details of the nine connection configurations are shown in Figure 1. Each specimen consisted of two precast panels and a connection region. The dimensions correspond to a prototype scale of the precast panels used for highrise construction. All parameters, such as steel details and concrete material, were kept constant. The dry pack mix consists of 2 parts concrete sand, 1 part normal portland cement, and approximately 0.5 parts water. The mix was placed and compacted into the 20 mm wide joint space between the two panels and the multiple shear keys.

The specimens subjected to pure cyclic shear loading were tested as shown in Figure 2(a). The test setup used in the cyclic flexure/shear experimental phase is shown in Figure 2(b). In both cases, the bottom wall panel of the specimen was fixed to the rigid floor of the structures lab by post-
tensioning against two abutments located at each end of the lower panel. The horizontal cyclic load was applied to the top concrete panel using a 1000 kN capacity MTS closed loop cyclic actuator by means of a push/pull-type loading yoke. The configuration of the top panel in the shear specimens and the loading yoke allowed application of a pure shear load to the connection. In the flexural specimens, the loading yoke was located across the top panel at 1800 mm above the joint region. This configuration provided a moment, \( M \), to shear, \( V \), ratio for the given connection length, \( L \), \((M/V.L)\) of 1.5 for all test specimens. This ratio is typical value for connection near the base of a 10-story wall building. The vertical axial load of 2 MPa normal to the joint was applied using an independent prestressing system designed to prevent any constraints in the direction of the applied shear and flexural loads as shown in Figures 2(a) and (b). The specimen was instrumented to measure the applied load and to monitor different response mechanisms: 1) Overall panel to panel displacement; 2) Local deformations across joint including interface panel to panel slip and rocking behaviour which is opening and closing of the joint interface; and, 3.) Reinforcement strains at the connection region.

The testing procedure started by applying the vertical normal load of 2 MPa followed by initial reading of all the instrumentation. For the cyclic pure shear tests, loading proceeded by 3 cycles at increasing load increment of 50 kN up to cracking and slipping of the connection. After cracking, the test was controlled by the specified slip across the joint region with 3 cycles at slip increment of 1 mm. The loading pattern used for the reversed cyclic flexural tests was by applying controlled series of quasi-static fully reversed cyclic loading pattern, three cycles at each level. Initially, the load was applied using 25\% increments of connection strength. Subsequently, the specimen was subjected to displacement cycles at multiples of the yield displacement until failure. Between each of these sets of large amplitude cycles, a service-amplitude cycle (60\% of the yield) was inserted to evaluate possible degradation of the connection. The test was terminated when the load carrying capacity of the precast connection dropped below 80\% of the maximum load reached.

**TEST RESULTS AND DISCUSSION**

**Cyclic Shear Program**

The cyclic pure shear test results suggest that the cyclic shear behaviour of the connections could be described in three distinct stages as shown in Figure 3(a) which gives typical shear resistance-slip behaviour. The first stage describes the perfectly elastic, stiff behaviour prior to initiation of slip between the dry pack grout and the panels. The second stage describes the inelastic behaviour after the initiation of slip. During this stage, the dry pack grout remains virtually intact and there may be some increase or decrease in shear resistance with increased slip magnitudes. The behaviour of the specimen during this stage is almost rectangular shear resistance-slip hysteresis loops, as shown in Figure 3(a). The shear key connection (SK) exhibited a dramatic increase in shear resistance with limited slip during this stage due to the presence of the shear keys. The third stage of behaviour is initiated by sudden and extensive crushing and spalling of the dry pack grout, as shown in Figure 4(a), accompanied by a significant reduction in shear resistance. After failure of the dry pack grout, the hysteresis loops stabilized at a reduced shear resistance and the dry pack grout was virtually ground into powder.

The relative performance of the five connection configurations under cyclic shear loading is illustrated in Figure 5. The influence of the contribution of the different connection configurations on the cyclic behaviour is rather obvious.
**Cyclic Flexure/Shear Program**

Typical overall behaviour is given by load versus top displacement hysteresis loops in Figure 3(b). The cyclic behaviour of the connection could be characterized by three distinct limit states depending on the degree of joint deterioration: (a) Phase I: initial behaviour as linear elastic, stiff behaviour without visible damage; (b) Phase II: post-yield behaviour without significant joint deterioration with "stable" hysteresis loops and minor damage without extensive crushing; and, (c) Phase III: post-yield behaviour with significant joint deterioration at onset of failure with significant reduction in the load carrying capacity under increased deformation. This stage was characterized by hinging at one end of the joint with rupture or pull-out from sleeve of vertical joint reinforcement at the other end, by deterioration (significant crushing) and spalling of drypack and panel concrete at both sides of the joint and major slip with large displacement at the top of the panel at the onset of failure with the exception of the specimen with shear keys and post-tensioned bar specimens. Typical failure pattern of the cyclic flexure/shear connections tested is shown in Figure 4(b).

The three phases of connection behaviour are illustrated in strength envelope plots of load versus displacement in Figures 6(a) and (b) for specimen series with mild steel reinforcement and post-tensioned reinforcement, respectively. The behaviour could be summarized as: (i) Both welded and mechanically spliced connection detail have similar overall response characteristics; (ii) Shear key connection configuration had consistently limited slip response compared to the plain surface joint; (iii) as designed the RT connection had a lower strength and ductility in comparison to the welded or spliced connection due to failure of the tube in shear; (iv) post-tensioned bar and strand connection had similar overall response characteristics; and, (v) the connection with partially or fully debonded bar showed the best performance in terms of strength, stiffness, ductility, and cumulative energy dissipation as will be discussed in the following paragraph.

The large connection deformability associated with rocking and slip allowed the panel system to withstand large cycles of deformation beyond first yield without reduction of the load up to failure. Connection details of the rebar series had an ultimate ductility around 5. Post-tensioned connections had ductility capacity of 6. Partial debonding of rebar produced a ductility more than 9; and with full debonding of post-tensioned bar, a ductility of 13 was achieved. Stiffness degradation was monitored by means of service load cycles inserted between each ductility level. Typical measured stiffness degradation at failure was 85%.

Energy dissipation per cycle is defined as the area enclosed by the load-displacement hysteresis curve, calculated by numerical integration using an inhouse computer program. Comparison of the cumulative energy dissipation for the specimens with rebar and post-tensioning is given in Figures 7(a) and 7(b). As can be seen in Figure 7(a), the partially unbonded mild steel reinforcement specimen had the best per cycle and cumulative energy dissipation characteristics in comparison to all other specimens in the same category. The energy dissipation capability per cycle of specimens with and without shear keys, were nearly identical. The cumulative energy dissipation for the plain connection specimen was higher as it failed at a higher ductility level. The welded versus mechanically spliced configuration had a higher per cycle and cumulative energy dissipation. The bolted connection configuration had a lower cumulative energy dissipation per cycle due to failure of the tube element by shear. The post-tensioned bar detail had a higher per cycle and cumulative energy dissipation in comparison to the post-tensioned strand configuration, as shown in Figure 7(b). The unbonded post-tensioned bar detail had similar per cycle energy dissipation but by far the best cumulative energy dissipation in comparison to all specimens tested.
CONCLUSION

Seventeen specimens with nine different connection configurations were tested under cyclic pure shear and cyclic flexure/shear loading conditions to investigate the various limit states cyclic behaviour of horizontal connections. The effect of different connection configurations were investigated. Based on the results of this study, the following conclusions could be drawn:

1. The effect of the cyclic loading was to cause crushing of the dry pack grout. This behaviour introduces an additional limit state beyond what was observed for specimens tested under static loading conditions. This may dramatically reduce the resistance of the connection, depending on the specific resistance mechanisms involved.

2. All connections tested were capable to withstand large nonlinear deformations well beyond first yield with very good energy absorption. Connections with bonded mild reinforcement had ductility around 5. Post-tensioned connections had ductility of 6.

3. The presence of shear keys in the horizontal connection enhances the shear capacity in comparison to the plain surface connection.

4. Debonding of continuity element across the connection significantly enhanced the response of the connection in terms of energy dissipation and ductility.

5. Seismic response up to a ductility of three could be resisted by all the tested configurations without any apparent damage to the precast wall connection. This level represents typical seismic demand for low to moderate seismicity.

ACKNOWLEDGEMENTS

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REFERENCES


Table 1. Test Program

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<td>a</td>
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<td>Drypack</td>
<td>Strand</td>
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<td></td>
<td>Dry packed shear keys</td>
<td>Large Keys</td>
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<td>II</td>
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a Static shear behavior was studied @ UoM and is published by P-CI Journal
b Static flexural behaviour only for the three types of continuity elements: rebars, strands, PT bars

Figure 1. Connection Configuration

Figure 2. Laboratory test setups: a) Cyclic Shear; and b) Cyclic Flexure/Shear Tests
Figure 3. Typical cyclic hysteretic behaviour: a) Shear and b) Flexure and shear specimens

Figure 4. Typical failure modes: a) Shear and b) Flexure and shear specimens
Figure 5. Comparison of strength envelopes for cyclic shear specimens

Figure 6. Comparison of strength envelopes for flexure: a) rebars, and b) post-tensioning specimens

Figure 7. Comparison of cumulative energy for flexure: a) rebars, and b) post-tensioning specimens