CONNECTIONS FOR

PRECAST CONCRETE LOAD-BEARING SHEAR WALL PANELS

M.L. Lau(1), S.H. Rizkalla(2), and K.A. Soudki(3)

ABSTRACT

Precast load-bearing shear wall panels are used extensively for high-rise construction because of 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 a four-year research program conducted to investigate the behaviour and the capacity of nine connection configurations used for precast load-bearing shear wall panels subjected to monotonic shear loading.

The different configurations include two types of dry-packed multiple shear keys, 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 connections which support hollow-core slab with and without post-tensioning. Two different levels of load normal to the connection were used to determine the effects of dead load.

Rational mathematical models were developed to predict the shear capacity of the connections at the maximum and ultimate limit states. These models were found to be in good agreement with the experimental results.

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

Behaviour of the connections typically used for precast concrete load-bearing shear wall panels greatly influences the structural integrity of the entire structure. Current design sources (1, 2, 3, 4) provide very limited information to describe the behaviour and design of such connections. The information available in the literature (5, 6, 7, 8, 9), which mainly describes the shear friction concept and the dowel action mechanism of initially cracked concrete, is not directly applicable to the behaviour of such connections.

This paper discusses the results of three phases of an experimental program undertaken to examine the behaviour of nine connection configurations. The first phase (10) included four types of connections: a dry-packed plain surface connection, a dry-packed connection with continuity reinforcement, and dry-packed connections with two types of mechanical shear connectors, in addition to the continuity bars. The second phase (11) included two different types of dry-packed multiple shear key configurations. The third phase (12) was the result of a recent innovation in horizontal connections for load-bearing shear wall panels using vertical post-tensioning tendons to replace the continuity bars. The strands pass through galvanized ducts from the top panel to the base of the structure.
While the panels are temporarily braced, the gap between panels which is necessary for alignment purposes, is dry-packed. After the erection of several stories, the tendons are post-tensioned, the ducts are grouted, and the temporary braces are removed. The testing included connections which support hollow-core slab with and without post-tensioning, and post-tensioning connections without hollow-core slab. All the connections were subjected to a monotonic shear load up to failure.

**OBJECTIVE AND SCOPE**

The primary objective of this paper is to investigate the limit states behaviour of nine connection configurations used for precast concrete load-bearing shear wall panels. Twenty-two prototype precast concrete shear wall panel specimens were used to test nine different connections currently used by the construction industry. The study attempted to identify the contribution of each component used for these types of connections. The results of the experimental program were used to refine and calibrate proposed rational mathematical models introduced to predict the strength of such connections before and after cracking.

**EXPERIMENTAL PROGRAM**

**Test Specimens**

A total of twenty-two specimens were tested. Overall dimensions of the precast panels used and the location of the mechanical strain gauges stations are shown in Figure 1. The dimensions correspond to a prototype scale of the precast panels used for highrise construction. The nine connection configurations tested in this study are as follows:
Type I: Dry pack only.
Type II: Dry pack and 25M continuity bars.
Type III: Dry pack, 25M continuity bars, and shear connectors Type A.
Type IV: Dry pack, 25M continuity bars, and shear connectors Type B.
Type V: Large-size dry-packed multiple shear keys.
Type VI: Small-size dry-packed multiple shear keys.
Type VII: Dry pack and hollow core slab.
Type VIII: Dry pack, hollow core slab and post-tensioning.
Type IX: Dry pack and post-tensioning.

Details of the above connections are shown in Figure 2.

Each specimen consisted of two precast panels, as shown in Figure 1. All parameters, such as steel details and concrete material, were kept constant. Prior to testing, specially-designed temporary steel brackets were used for specimens with dry pack only. 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 mixture was of a dry consistency, which allows the tamping and compaction of the material.

For phase III specimens, the hollow-core slab rests on "Korolath" bearing pads on the bottom panel. The cores of the hollow-core slab and the gap between the ends of two slabs are filled with a flowable concrete fill. The gap between the hollow-core and the top panel is dry-packed. Seven days after drypacking, the strands were post-tensioned and the ducts were filled using an expansive grout.
Testing Apparatus and Procedure

Each specimen was positioned vertically into the testing machine, as shown in Figure 3. The joint centerline of the specimen was aligned with the vertical centerline of the top and bottom heads of the testing machine. To study the dead load effect, two levels of preload applied normal to the joint equivalent to 2 MPa and 4 MPa were used for Phases I and II specimens, and 4 MPa and 8 MPa for Phase III specimens. A post-tensioning scheme consisting of four hydraulic jacks, Dywidag bars and a series of plates designed to allow deformation of the joint were used to measure the preload, applied normal to the joint, as clearly shown in Figure 3. An additional post-tensioning scheme was used at the outer edges of each panel to prevent premature failure of the panel at the loading zones.

The testing procedure was started by applying the normal preload to the designed level followed by initial reading of all the instrumentation. The testing machine was used to apply the vertical shear load in increments of 100 kN. At each increment, readings of all instrumentation were recorded followed by marking of the cracks. The experiment was terminated after extensive deformation and a significant drop in the load-carrying capacity was observed. Descriptions of the tested specimens of the three phases are given in Table 1.

TEST RESULTS AND DISCUSSION

Typical failure patterns of the various connections tested in Phases I, II and III are shown in Figures 4a, 4b, 4c, and 4d. In general, the predominant mode of failure was due to slip along the drypack to panel interface. Cracking of the hollow core at the
bottom of the connection was observed in Phase III, as shown in Figure 4d. Typical load-slip behaviour of the connections tested in Phase I under a preload level of 2 MPa and 4 MPa are shown in Figures 5 and 6, respectively. The behaviour of the specimens tested in Phase II under the same preload levels are shown in Figures 7 and 8. Figures 9, 10, 11, and 12 show the load-slip behaviour of the connections tested in Phase III under the effect of various loading conditions.

The test results of Phase I indicate that regardless of the mechanical connectors used, the cracking strength of the connection depends mainly on the bond strength at the dry pack concrete interface. The residual shear, which represents the ultimate capacity of the connection, is related to the level of the load normal to the joint, the dowel action of the continuity bars, and the resistance of the mechanical connectors. Thus, the ultimate shear resistance, $V_u$, of these connections can be predicted using the following proposed model:

$$V_u = \mu \sigma_n A_c + \frac{f_y}{\sqrt{3}} + V_w$$  \hspace{1cm} (1)

where

- $\mu$ = friction coefficient factor, 0.7 is proposed
- $\sigma_n$ = compressive stresses normal to the connection, psi
- $A_c$ = cross-sectional area of the connection, in$^2$
- $f_y$ = yield strength of the continuity bars, psi
- $A_b$ = area of the continuity bars, in$^2$
- $V_w$ = shear strength of the weld, lb

The predicted ultimate shear resistance based on the proposed model was found to be in good agreement with the measured values of Phase I, as shown in Figure 13.
The test results of Phase II indicate that using multiple shear keys will enhance the maximum shear capacity by as much as 60 percent in comparison to the plain surface connections at the same level of preload. The behaviour of these connections suggests that the shear capacity depends mainly on the strength of the weaker material within the vicinity of the connection and the level of the load normal to the connection. The difference in multiple shear key configurations used in this study had no measurable effect on the ultimate shear load capacity of the connection. It should be noted that the strength of the dry pack used in Phase II is considerably lower than the dry pack strength used in Phase I.

Based on the observed behaviour after cracking, the maximum shear load, $V_m$, of the multiple shear key connection is mainly governed by the compressive strength of the struts between the diagonal cracks and the shear friction resistance along the slip surface, as illustrated in Figure 14. Therefore, the predicted maximum shear load, $V_m$, can be expressed in terms of these two components as follows:

$$V_m = V_{me} + V_{mf}$$

where $V_{me}$ is the shear resistance of the strut mechanism and $V_{mf}$ is the shear friction resistance along the slip surface. In this analysis, the shear wall panels are assumed to act as rigid bodies connected by $n-1$ struts, where $n$ is the number of shear keys. For the three keys in Figure 11(a), the struts are shown schematically in Figure 14(b). The compressive strength of the cracked drypack, $f_{cz}$, could be evaluated using Collins and Mitchell’s equation (3):

$$f_{cz} = \frac{f_{y}}{0.8 + 170\epsilon_1}$$

(3)
where $\varepsilon_t$ is the average maximum principal tensile strain in the drypack at cracking. For the multiple shear key connections tested in this study, the measured strain, $\varepsilon_t$, varied between 0.0026 and 0.004 strain. Using the maximum measured strain value of 0.004, $f_{c2}$ may be taken as 0.67 $f'_c$ for these types of connections, where $f'_c$ is the compressive strength of the drypack. Thus, the shear resistance of the strut mechanism, $V_{mc}$, may be estimated as:

$$V_{mc} = (n-1)f_{c2}A_{cs}\sin\alpha$$

(4)

where $A_{cs}$ is the average cross-sectional area of the diagonal portion of the strut and $\alpha$ is the inclination of the diagonal portion of the strut to the horizontal.

Representing the connection by a rectangular strip, the distribution of forces at the connection, including the shear friction resistance, $V_{mf}$, provided by slip along the drypack-panel interface and along the diagonal cracks, is shown in Figure 14(c). The shear friction resistance, $V_{mf}$, may be evaluated as:

$$V_{mf} = \mu(\sigma_n - \frac{(n-1)f_{c2}A_{cs}\cos\alpha}{A_e})A_e$$

(5)

where $\mu$ is the friction coefficient and $\sigma_n$ is the compressive stress normal to the connection. Therefore, the maximum shear capacity after cracking, $V_m$, according to Eq. (2), can be estimated as:

$$V_m = (n-1) f_{c2} A_{cs} \sin\alpha + \mu \left(\sigma_n - \frac{(n-1)f_{c2}A_{cs}\cos\alpha}{A_e}\right) A_e$$

(6)

Assuming a value of 0.6 for the friction coefficient, the predicted maximum shear capacities
according to Eq. (6) are in good agreement with the measured values of Phase II, as shown in Figure 15.

The ultimate shear resistance of the multiple shear key connection mainly depends on the level of load normal to the connection, and the bearing stresses and shear friction along the slip surfaces. As discussed earlier, the configuration of the shear keys considered in this investigation was found to have an insignificant effect on the shear capacity. Using a linear regression analysis, the following model was developed to predict the ultimate shear resistance of the multiple shear key connections in terms of the bearing and shear resistances:

\[ V_u = 2.4 \sqrt{f_s} A_{ck} + 0.5 \sigma_n A_c \]  

(7)

where \( A_{ck} \) is the cross-sectional area for the portion of the connection covered by the shear keys, and \( A_c \) is the cross-sectional area for the entire length of the connection. \( A_{ck} \) is equal to \( A_c \) if the shear keys cover the entire length of the connection.

The predicted ultimate shear resistance, using Eq. (7), is compared to the measured values of Phase II in Figure 16.

The test results in Phase III indicate that the increase in the maximum and ultimate shear capacities is directly proportional to the increase of the load normal to the connection as shown in Figures 9 and 10. At the low stress level, 4 MPa, normal to the connection, the presence of hollow-core slab had no or little effect on the behaviour or the capacity of the post-tensioned connection, as shown in Figure 11. However, at higher levels of stress normal to the connection, the presence of hollow-core slab significantly reduced the maximum and ultimate shear capacities of the connection, as shown in Figure 12.
Based on the observed behaviour after cracking, the failure mechanism in horizontal connections with hollow-core slab appears to be controlled either by the shear capacity of the hollow-core slab, as in those specimens tested at higher load levels normal to the connection, or by the shear friction resistance of the connection, as in those specimens tested at the lower load level.

The maximum shear capacity could be predicted as the lesser of that determined by the shear friction model, $V_n$, equation (8a), and the cracking capacity of the hollow-core, $V_h$, equation (8b).

$$V_i = \mu \sigma_A$$  \hspace{1cm} (8a)

$$V_n = \frac{2}{3} (A_{c1}F_{t1} + A_{c2}F_{t2})$$  \hspace{1cm} (8b)

$$F_{t1} = \sqrt{f_{t1}(f_{t1} + \sigma_{n1})}$$

$$F_{t2} = \sqrt{f_{t2}(f_{t2} + \sigma_{n2})}$$

The shear friction resistance, $V_n$, can be predicted using the shear friction model which is related to the area of the concrete interface, $A_c$, and the coefficient of friction, $\mu$, of the dry pack to panel interface. A coefficient of friction of 0.7 is proposed based on the test results of Phase 1 investigation.

The cracking capacity of the connection, $V_h$, based on the capacity of the hollow-core slab with concrete fill, can be predicted using the areas of the hollow-core and concrete fill in contact with the drypack, $A_{c1}$ and $A_{c2}$, respectively. The magnified tensile capacities of the concrete of the hollow-core and the concrete fill, $F_{n1}$ and $F_{n2}$, respectively, are based on the tensile strength of each material, $f_{n1}$ and $f_{n2}$, and the compressive normal stresses, $\sigma_{n1}$ and $\sigma_{n2}$, including the effect of prestressing. Figure 17 shows the distribution
of vertical load and computation of various parameters as stated above.

Figure 18 compares the maximum shear capacity of all nine specimens tested in Phase III and the capacity as predicted by the proposed model. The predicted maximum shear capacities are in good agreement with the experimental results.

The ultimate shear capacity of horizontal connections with hollow-core could be predicted as the lesser of that determined by the shear friction model, \( V_r \), equation (8a), and the ultimate capacity of the hollow-core, \( V_{hu} \), equation (9):

\[
V_{wu} = \frac{2}{3} (A_{wu}F_{wu} + A_{w2}F_{w2})
\]

\[
F_{wu} = \sqrt{f_n(f_{n1} + \sigma^1_{n1})}
\]

\[
F_{w2} = \sqrt{f_n(f_{n2} + \sigma^2_{n2})}
\]

Where \( A_{wu} \) is the summation of the area of the webs at mid-height of the hollow-core beneath the contact surface area \( A_{c1} \). For the given geometry of the hollow-core tested in this investigation, \( A_{wu} = A_{c1}/4 \).

In this model, the contribution from the centre concrete fill remains unchanged while the contribution from the hollow-core slabs are modified to reflect the loss of bond. The area of the hollow-core in contact with the dry pack, \( A_{c1} \), is reduced by a factor of four. However, the normal stress, \( \sigma^1_{n1} \), is increased, also by a factor of four, due to the reduction of the contact area at ultimate.

Figure 19 compares the measured and the predicted ultimate shear capacity of all nine specimens tested in Phase III. The comparison suggests that the proposed model
provides a conservative lower bound for the ultimate shear capacity of horizontal connections with hollow-core slab.

CONCLUSION

Twenty-two specimens with nine different connection configurations were tested under monotonic shear loading conditions to investigate the various limit states behaviour of horizontal connections. The effect of connection configuration, load normal to connection, presence of hollow-core, and post tensioning were investigated.

Based on the results of this study, the following conclusions could be drawn:

1. An increase in the level of load normal to the connection increases the maximum shear capacity of this connection.

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

3. The difference in the shear key configurations considered in this study had insignificant effect on the behaviour or capacity of the connection.

4. The shear capacity depends mainly on the strength of the weaker material within the vicinity of the connection.

5. The maximum shear capacity of the connection with hollow-core slab is governed by the lower magnitude of the shear friction capacity and the hollow-core shear capacity.

6. The ultimate shear capacity of the connection with hollow-core slab is based on complete loss of bond. The predicted values provide a conservative lower bound.
7. Replacing the continuity bars in the shear wall panels with vertical post-tensioning for connecting the panels enhances the shear capacity, in addition to its economical advantages.

8. 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.

ACKNOWLEDGEMENTS

This experimental program was conducted at the University of Manitoba, Winnipeg, Canada, with financial assistance from Con-Force Structures Ltd., Winnipeg, Manitoba, and the Natural Sciences and Engineering Research Council of Canada (NSERC).
REFERENCES

1. American Concrete Institute (ACI), "Building Code Requirements for Reinforced Concrete", Detroit, Michigan, 1983.


Table 1. Overall test parameters.

<table>
<thead>
<tr>
<th>Dry Pack Configuration</th>
<th>Load normal to connection MPa</th>
<th>Type of connection</th>
<th>Specimen mark</th>
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<td>SP11</td>
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<tr>
<td></td>
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<td>I</td>
<td>SP21</td>
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<td></td>
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<td>SP22</td>
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Figure 1. Overall dimensions and average strain gauge locations of typical test specimen.
Figure 2. Details of the nine connections considered in this study.
Type VIII  Dry pack grout, hollow-core slab, and post-tensioning

Type VII  (same as above without post-tensioning)

Type IX    (same as above, without hollow-core)

Figure 2.  (Continued) Details of the nine connections considered in this study.
Figure 3. Test set-up.
Figure 4a. Typical failure of connections I and II.
Figure 4b. Typical failure of connections III and IV.
Figure 4c. Typical failure of connections V and VI.
Figure 4d. Typical cracking of hollow-core at ultimate.
Figure 5. Load-displacement of the connections tested in Phase I under preload of 2 MPa.

Figure 6. Load-displacement of the connections tested in Phase I under preload of 4 MPa.
Figure 7. Load-displacement of the connections tested in phase II under preload of 2 MPa.

Figure 8. Load-displacement of the connections tested in phase II under preload of 4 MPa.
Figure 9. Effect of the load normal to the connection.

Figure 10. Effect of post-tensioning.
Figure 11. Effect of the presence of hollow-core slab at low stress level normal to the connection.

Figure 12. Effect of the presence of hollow-core slab at high stress level normal to the connection.
Figure 13. Relationship between the measured and predicted ultimate shear capacity of the connections for Phase I.
Figure 14. Maximum shear strength model.
Figure 15. Comparison of predicted to measured maximum load for Phase II.

Figure 16. Comparison of predicted to measured ultimate load for Phase II.
In this investigation:

\[ b_1 = b_2 = 50 \text{ mm} \]
\[ A_{c1} = 2 [50 \times 1200] = 12000 \text{ mm}^2 \]
\[ A_{c2} = 50 \times 1200 = 6000 \text{ mm}^2 \]
\[ A_{cu} = 2 [50 \times \Sigma \text{ web}] \\
= 2 [50 \times (5 \times 40 + 2 \times 50)] = 3000 \text{ mm}^2 \]

For Specimen 5HP8:

\[ \sigma_n = 8 + 1.2 \]
\[ = 9.2 \text{ MPa} \]
\[ \sigma_{n1} = 6.5 \text{ MPa} \]
\[ \sigma_{n2} = 14.6 \text{ MPa} \]
\[ \sigma_{n1}^{'} = A_{c1}/A_{cu} \sigma_{n1} \\
= 12000/3000 \sigma_{n1} \\
= 26 \text{ MPa} \]

Figure 17. Distribution of vertical load and dimensions of typical horizontal connection with hollow-core slab
Figure 18. Relationship between the measured and predicted maximum shear capacity of the connection for Phase III.

Figure 19. Relationship between the measured and predicted ultimate shear capacity of the connection for Phase III.