Design Considerations for the Connection of Precast Concrete Structures

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SUMMARY

The structural integrity of precast concrete buildings depends mainly on the connections between the precast structural components. Large precast concrete panels are used extensively in North America for low, medium, and highrise construction due to their ease and speed of assembly, and high quality control.

This paper presents the results of twenty-two prototype specimens tested to investigate the behaviour and the capacity of typical connections used for precast loadbearing shear wall panels subjected to shear loading.

The different configurations include dry-packed for plain surface panels, two sizes of dry-packed multiple shear keys, dry-packed plain surface and continuity bars with and without mechanical shear connectors, and post-tensioned connections. Shear walls supporting hollow-core floor slabs were also included. Different levels of load normal to the connection were used to simulate the effects of gravity and permanent loading conditions.

Test results were used to refine and calibrate rational mathematical models developed to predict the maximum shear capacity and the nominal shear strength of the connection. Various failure mechanisms are presented and discussed.

The findings of this investigation were implemented in the design of three 32-storey apartment buildings in Winnipeg, Manitoba, and 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 (ACI, 1989; PCI, 1985; CSA, 1984; CPCII, 1987) provide very limited information to describe the behaviour and design of such connections. The information available in the literature (Birkeland and Birkeland, 1966; Mast, R.F. 1968; Holbeck and Abraham, 1969; Mattock and Hawkins, 1972; Mattock, 1974), 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 as given in Table 1. The first phase (Föerster et al., 1989) 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 (Serrette et al., 1989) included two different types of dry-packed multiple shear key configurations. The third phase (Hutchinson et al., 1991) included vertical post-tensioning tendons to replace the continuity bars. Testing included shear walls supporting hollow core slabs. All the connections were subjected to a monotonic shear load up to failure. Three different levels of load normal to the connection were used to simulate the effects of gravity and permanent loading conditions.

RESEARCH SIGNIFICANCE

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 identified 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. 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.

The configuration of the post-tensioned connections supporting hollow core slabs and the dimensions of the precast panels used for typical specimens are shown in Figure 1. Details of the other six 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 "Korolith" 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 effect of the gravity load, three levels of preload applied normal to the joint were used. 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.

**TEST RESULTS AND DISCUSSION**

Selected 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 4 MPa is shown in Figure 5. The behaviour of the specimens with multiple shear keys, tested in Phase II under the same preload level is shown in Figure 6. The load-slip behaviour of the post-tensioned connections, with and without the hollow core slab, subjected to preload levels of 8 MPa is shown in Figure 7.

The test results of Phase I indicate that the cracking strength of the connection depends mainly on the bond strength at the dry pack concrete interface. The residual shear, which represents the nominal shear strength 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 nominal shear strength, $V_n$, of these connections can be predicted using the following proposed model:

$$V_n = \mu \sigma_s A_c + A_s f_y \sqrt[3]{3} + V_w$$

where

- $\mu$ = friction coefficient factor, 0.7 is proposed
- $\sigma_s$ = 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
- $A_s$ = area of the continuity bars, in$^2$
- $V_w$ = shear strength of the weld, lb

The predicted nominal 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 8.

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 capacity, $V_m$, 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 9. Therefore, the predicted maximum shear capacity, $V_m$, can be expressed in terms of these two components as follows:

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

where $V_{mr}$ 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 9(a), the struts are shown schematically in Figure 9(b). The compressive strength of the cracked drypack, \( f'_{ck} \), could be taken as 0.67 \( f'_{ck} \) for these types of connections, where \( f'_{ck} \) is the compressive strength of the drypack (Serrette et al., 1989). Thus, the shear resistance of the strut mechanism, \( V_{s} \), may be estimated as:

\[
V_{s} = (n-1)f'_{ck}A_{s} \sin \alpha
\]  

(3)

where \( A_{s} \) 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, \( V_{fr} \), provided by slip along the drypack-panel interface and along the diagonal cracks, is shown in Figure 9(c). The shear friction resistance, \( V_{fr} \), may be evaluated as:

\[
V_{fr} = \mu (\sigma_{n} - \frac{\sigma_{s} \cos \alpha}{A_{s}}) A_{s}
\]  

(4)

where \( \mu \) is the friction coefficient and \( \sigma_{n} \) is the compressive stress normal to the connection. Therefore, the maximum shear capacity after cracking, \( V_{u} \), according to Eq. (2), can be estimated as:

\[
V_{u} = (n-1) f'_{ck} A_{s} \sin \alpha + \mu (\sigma_{n} - \frac{(n-1)f'_{ck} A_{s} \cos \alpha}{A_{s}}) A_{s}
\]  

(5)

The predicted maximum shear capacities according to Eq. (6) were in good agreement with the measured values of Phase II, using a value of 0.6 for the friction coefficient.

The nominal shear resistance, \( V_{nak} \), of the multiple shear key connection mainly depends on the load of shear 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 nominal shear resistance of the multiple shear key connections in terms of the bearing and shear resistances:

\[
V_{nak} = 2.4 \sqrt{f'_{ck} A_{n}} + 0.5 \sigma_{n} A_{s}
\]  

(6)

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

The predicted nominal shear resistance, using Eq. (6), was in excellent agreement with the measured values of Phase II, as shown in Figure 10.

The test results in Phase III indicate that the increase in the maximum shear capacity and nominal shear strength is directly proportional to the increase in the load normal to the connection. 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. However, at higher levels of stress normal to the connection, the presence of hollow-core slab significantly reduced the maximum and nominal shear capacities of the connection.

The maximum shear capacity could be predicted as the lesser of that determined by the friction resistance model, \( V_{f} \), Eq. (7), and the cracking capacity of the hollow-core, \( V_{cr} \), Eq. (8b).

\[
V_{f} = \mu \sigma_{n} A_{s}
\]  

(7a)

\[
V_{n} = \sqrt[3]{(A_{n}F_{fi} + A_{c}F_{ci})}
\]  

(7b)

\[
F_{fi} = \sqrt{f_{i}(f_{i} + \sigma_{n})}
\]

\[
F_{ci} = \sqrt{f_{c}(f_{c} + \sigma_{n})}
\]
The shear friction resistance, \( V_f \), can be predicted using the shear friction model which is related to the area of the concrete interface, \( A_s \), 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 I investigation.

The cracking capacity of the connection, \( V_c \), 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, \( A_{cf} \), and \( A_{at} \), respectively. The magnified tensile capacities of the concrete of the hollow-core and the concrete fill, \( F_{tc} \) and \( F_{tc} \), respectively, are based on the tensile strength of each material, \( f_{tc} \) and \( f_{tc} \). The compressive normal stress, \( \sigma_{at} \), is the portion of the total normal stress, including the effect of prestressing acting on the hollow core slab portion of the connection. \( \sigma_{at} \) is the normal stresses acting on the concrete fill portion of the connection (Hutchinson et al., 1991).

The nominal shear strength of horizontal connections with hollow-core could be predicted as the lesser of that determined by the shear friction model, \( V_f \), Eq. (7a), and the ultimate capacity of the hollow-core, \( V_{hs} \), Eq. (8):

\[
V_{hs} = \frac{1}{3} (A_{cf}F_{tc} + A_{at}F_{tc})
\]  

(8)

\[
F_{tc} = \sqrt{f_{tc}(f_{tc} + \sigma_{at})}
\]

\[
F_{tc} = \sqrt{f_{tc}(f_{tc} + \sigma_{at})}
\]

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

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, \( A_{at} \), is reduced by a factor of four. However, the normal stress, \( \sigma_{at} \), is increased, also by a factor of four, due to the reduction of the contact area at ultimate.

The comparison between the measured and predicted nominal shear strength suggests that the proposed model provides a conservative lower bound for the nominal shear capacity of horizontal connections with hollow-core slab as shown in Figure 11.

CONCLUSION

Twenty-two specimens with nine different connection configurations were tested under monotonic shear loading conditions to investigate the various limit state 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 nominal shear strength 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.
ACKNOWLEDGEMENTS
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REFERENCES

Rizkalla et al., "Design Considerations for ... " 6
### 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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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 1. Overall dimensions of typical test specimens of post-tensioned connection supporting hollow core slab type VIII.
Figure 2. Details of the nine connections considered in this study.
Figure 3. Test set-up
Figure 4. Typical failure of the connection.
Figure 4. Typical failure of the connection (cont'd)
Figure 5. Load-displacement of the connections tested in Phase I under preload of 4 MPa

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

Figure 7. Effect of the presence of hollow-core slab at high stress level normal to the connection
Figure 8.  Relationship between the measured and predicted nominal shear strength of the connection for Phase I

Figure 9.  Force mechanism for multiple shear connections
Figure 10. Relationship between measured and predicted nominal shear strength of the connection for Phase II

Figure 11. Relation between measured and predicted nominal shear strength of the connection for Phase III