CONNECTIONS FOR PRECAST CONCRETE PANELS
USED FOR HIGHRISE STRUCTURES

S.H. Rizkalla(1), J. West(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 five-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 paper also includes results of dry-packed connections subjected to reversed cyclic loads equivalent to earthquake loads.

Connection 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. Three different levels of load normal to the connection were used to determine the effects of gravity 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.

1 Professor, Civil Engineering Department, University of Manitoba
2 Graduate Student, Civil Engineering Department, University of Manitoba
3 Doctoral Candidate, Civil Engineering Department, University of Manitoba
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, 1983; PCI, 1983; CSA, 1984; CPCI, 1987) provide very limited information to describe the behaviour and design of such connections. The information available in the literature (Birkeland, 1966; Mast, 1968; Hofbeck, 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 four phases of an experimental program undertaken to examine the behaviour of nine connection configurations. The first phase (Foerster 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) 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. Test results of a dry-packed connection, tested in the fourth phase, subjected to cyclic reversed loading conditions equivalent to earthquake loads, are also presented.

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-three 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-three specimens were tested. Overall dimensions of the precast panels used in the first three phases are shown in Figure 1(a). Configuration of the panel subjected to
reversed cyclic loading is shown in Figure 1(b). 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. 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**

For the first three phases, specimens were positioned vertically into the testing machine, as shown in Figure 3(a). 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 gravity 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 were used to apply and measure the preload, normal to the joint, as clearly shown in Figure 3. The loading system was designed to allow deformation of the connection parallel to the applied shear load. An additional post-tensioning scheme was used at the outer edges of each panel to prevent premature failure of the panel. The specimen subjected to cyclic loading, in Phase IV, was tested in horizontal position, as shown in Figure 3(b). In this case, the bottom panel was fixed to the strong floor of the structural laboratory, while the cyclic shear loading was applied to the top panel. A preload level of 4 MPa was applied normal to the joint to account for the gravity loads.

The testing procedure started by applying the normal preload to the designed level followed by initial reading of all the instrumentation. For the first three phases, 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. The loading pattern used for the reversed cyclic loading in Phase IV is shown in Figures 4(a) and (b). Initially, the load was increased by a load increment of 50 kN for a total of 30 cycles up to cracking and slipping of the connection, as shown in Figure 4(a). After cracking, the test was controlled by specified slip across the joint region measured by two LVDT's mounted on either
side of the connection, as shown in Figure 4(b). Descriptions of the specimens tested in this research program are given in Table 1.

TEST RESULTS AND DISCUSSION

Typical failure patterns of the various connections tested in Phases I, II, III, and IV are shown in Figures 5a, 5b, 5c, 5d, and 5e. 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 5d. For the specimen subjected to cyclic loading tested in Phase IV, extensive crushing and spalling of the dry pack grout occurred after slip of ±6 mm, and the thickness of the grout was substantially reduced, as shown in Figure 5e. Typical load-slip behaviour of the connections tested in Phase I under a preload level of 4 MPa is shown in Figure 6. The behaviour of the specimens tested in Phase II under the same preload level is shown in Figure 7. The load-slip behaviour of the post-tensioned connections tested in Phase III, with and without the hollow core slab, subjected to a preload level of 8 MPa, is shown in Figure 8. Typical load-slip hysteresis loops for the specimen tested in Phase IV are shown in Figure 9.

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 drypack 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:

$$V_u = \mu \sigma_n A_c + A_s f_y \sqrt{A_b} + V_w$$

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 10.

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

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\[ V_m = V_{mc} + V_{mf} \]  

where \( V_{mc} \) 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 11(b). The compressive strength of the cracked drypack, \( f'_{c2} \), could be evaluated using Collins and Mitchell's equation (3):

\[ f'_{c2} = \frac{f'_{c}}{0.8 + 170\varepsilon_1} \]  

where \( f'_{c1} \) 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, \( \varepsilon_1 \), varied between 0.0026 and 0.004 strain. Using the maximum measured strain value of 0.004, \( f'_{c1} \) may be taken as 0.67 \( f'_{c2} \) for these types of connections, where \( f'_{c2} \) is the compressive strength of the drypack. Thus, the shear resistance of the strut mechanism, \( V_{mc} \), may be estimated as:

\[ V_{mc} = (n-1)f'_{c2}A_s \sin \alpha \]  

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, \( V_{mf} \), provided by slip along the drypack-panel interface and along the diagonal cracks, is shown in Figure 11(c). The shear friction resistance, \( V_{mf} \), may be evaluated as:

\[ V_{mf} = \mu (\sigma_n - \frac{(n-1)f'_{c2}A_s \cos \alpha}{A_s}) A_e \]  

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:

\[ V_m = (n-1)f'_{c2}A_s \sin \alpha + \mu (\sigma_n - \frac{(n-1)f'_{c2}A_s \cos \alpha}{A_s}) A_e \]  

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.

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:
where $A_{ck}$ is the cross-sectional area for the portion of the connection covered by the shear keys, and $A_i$ is the cross-sectional area for the entire length of the connection. $A_{ck}$ is equal to $A_i$ 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 12.

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. 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 ultimate shear capacities of the connection.

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 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 friction model, $V_f$, equation (8a), and the cracking capacity of the hollow-core, $V_h'$, equation (8b).

\[
V_f = \mu \sigma_c A_c \quad (8a)
\]

\[
V_h' = \frac{1}{2} \left( A_{c1} F_{u1} + A_{c2} F_{u2} \right) \quad (8b)
\]

The friction resistance, $V_f$, can be predicted using the 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 I 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 dry pack, $A_{c1}$ and $A_{c2}$, respectively. The magnified tensile capacities of the concrete of the hollow-core and the concrete fill, $F_{u1}$ and $F_{u2}$, respectively, are based on the tensile strength of each material, $f_{u1}$ and $f_{u2}$, and the compressive normal stresses, $\sigma_{n1}$ and $\sigma_{n2}$, including the effect of prestressing.

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

\[
V_{hu} = \frac{1}{3} \left( A_{c1} F_{wu} + A_{c2} F_{w2} \right) \quad (9)
\]
Where $A_{ac}$ is the summation of the area of the webs at mid-height of the hollow-core beneath the contact surface area $A_{cl}$. For the given geometry of the hollow-core tested in this investigation, $A_{ac} = \frac{A_c}{4}$.

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

Figure 13 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.

The behaviour of the dry-packed connection subjected to reversed cyclic loading, tested in Phase IV, could be described according to the following distinct stages shown in Figure 14. In the first stage, prior to cracking, the connection behaved elastically without any measurable slip. The connection cracked after completion of 30 cycles with cracking strength as 25 percent less than the value measured in Phase I (10). The reduction could be attributed to the effect of the reversed cyclic loading conditions and possible variation of the materials. Testing of additional specimens, currently in progress, should provide additional information to verify and quantify this conclusion.

After cracking and loss of bond between the dry pack and the panel interface, the connection maintained a fairly constant shear capacity, as shown in Stage II of Figure 14. The load-slip hysteresis during this stage is shown in Figure 9. During this stage, the grout remained relatively intact, with some minor vertical cracks at the two ends of the connection. The shear strength at this stage can be predicted very well using Eq. (1) with a coefficient of friction of 0.7, for a total of 55 cycles. After 55 cycles and an applied slip of $\pm 6$ mm, extensive crushing and spalling of the dry pack was observed. The deterioration of the grout resulted in a 15 percent reduction in the shear capacity, as shown in Stage III of Figure 14. The connection capacity remained relatively constant even after 75 percent reduction of the grout thickness in this stage, as shown in Figure 5(e). The connection strength can be accurately predicted also by using Eq. (1) using a coefficient of friction of 0.7.

The above results suggest that the equation proposed for monotonic loading conditions could also be applied to earthquake loading conditions. However, to build in a similar level of reliability used in the monotonic models, it is recommended to use a friction coefficient of 0.6 for dry-packed connections for seismic zones.

Research is currently in progress to examine the behaviour of other types of connection configurations under reversed cyclic loading conditions.
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. Preliminary test results indicated that the equation proposed for monotonic loading could be used for earthquake loading using a coefficient of friction of 0.6.
9. 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

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REFERENCES

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


Table 1. Overall test parameters.

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Figure 1(a). Specimen configuration for Phases I, II, and III.

Figure 1(b). Specimen configuration for Phase IV.
Figure 2. Details of the nine connections considered in this study.

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 3(a). Phase I, II, and III test setup.

Figure 3(b). Phase IV test setup.
Load Control is used until cracking occurs. The test is continued under slip control.

Figure 4(a). Cyclic loading history - load control.

Figure 4(b). Cyclic loading history - slip control.
Figure 5. Typical failure of the connection.
Figure 5. Typical failure of the connection.
Figure 6. Load slip behaviour of the connections tested in Phase I under a preload of 4 MPa.

Figure 7. Load slip behaviour of the connections tested in Phase II under a preload of 4 MPa.

Figure 8. Load slip behaviour of the connections tested in Phase III under a preload of 8 MPa.
Figure 9. Load slip behaviour of Phase IV specimen.

Figure 10. Relationship between the measured and predicted ultimate shear strength of the connections in Phase I.
Figure 11. Force mechanism for multiple shear connections.

Figure 12. Relationship between the measured and predicted ultimate shear strength of the connections in Phase II.

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Figure 13. Relationship between the measured and predicted ultimate shear strength of the connections tested in Phase III.

Figure 14. Average strength envelope of the Phase IV specimen.