during the past decade, use of precast loadbearing shear wall panels for high rise construction has become very popular due to the high quality control achieved at the precast concrete manufacturing plant and the ease of panel assembly and erection at the project site (Fig. 1).

In the structure, the precast panels are tied together using continuity reinforcement bars and mechanical shear connectors. The gap between the panels, which is required for alignment, is normally filled with a dry pack grout. This grouting system eliminates the use of falsework and minimizes the use of temporary supports.

Current design sources\(^1\) provide only limited information about the behavior and ultimate shear carrying capacity of the connections described in this paper. The information available in the literature\(^2\) mainly describes the shear friction mechanism and the dowel action of initially cracked concrete to concrete surfaces. The results of such studies cannot be directly applicable to the pre-described connections due to the presence of the dry pack grout layer between the two concrete surfaces.
RESEARCH SIGNIFICANCE

The main objective of this study is to investigate the behavior of different types of connections typically used in practice for precast concrete load-bearing shear wall panels. The study attempts to identify the contribution of each component used for these types of connections at various limit states. 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 ten specimens were tested. Dimensions of all the precast panels were identical, with a thickness of 200 mm (7\% in.) and overall dimensions of 1660 mm and 1290 mm (65\% and 50\% in.), as shown in Fig. 2. The dimensions correspond to a prototype scale of the precast panels typically used for high rise construction. A typical reinforcement detail of the panel used for all the specimens is shown in Fig. 3. The five connection configurations used in this program are as follows:

- Type I: Dry pack grout only.
- Type II: Dry pack grout and 25M (No. 8) continuity bars.
- Type III: Dry pack grout, 25M (No. 8) continuity bars, and shear connectors Type A.
- Type IV: Dry pack grout, 25M (No. 8) continuity bars, and shear connectors Type B.
- Type V: Shear keys with dry pack grout.

Details of the above connections are shown in Fig. 4.

The average compressive strength of the normal weight concrete and the dry pack grout used for each specimen were determined using standard 150 x 300 mm (6x12 in.) concrete cylinders and 75 mm (3 in.) cubes, respectively. The concrete, supplied by a local ready-mix concrete company, had a maximum aggregate size of 14 mm (\% in.) and an average slump of 75 mm (3 in.). The reinforcement bars were Grade 400W (58 ksi) and the shear connection plates were 300W (43 ksi) steel, according to CSA CAN3-S16.1-M84.9.

Each specimen consisted of two precast panels as shown in Fig. 5. All parameters, such as steel details and concrete material, were kept constant. The two panels were connected after 28 days using the various connection details previously described. Specially designed temporary steel brackets were used for specimens with dry pack grout only.

The dry pack grout mix consists of two parts concrete sand, one part normal portland cement and approximately one-half part water. The mix was placed and compacted into the 20 mm (\% in.) wide joint space between the two panels. The mixture was of a dry con-
Fig. 1. Precast high rise building.

Consistency, which allowed for tamping and compaction of the material.

Electric resistance strain gages were used to measure the strains of the continuity bars and the shear connector plates. The average strains of the concrete and the dry pack were measured using mechanical strain gages at various locations, as shown in Fig. 6 for a typical specimen. Linear variable differential transducers were also used to monitor the deformation parallel and perpendicular to the joint.

Figs. 2 and 3 (opposite page) show the panel configuration and the typical reinforcement of the panel.
Fig. 2. Panel configuration.

25.4 mm = 1 inch

Fig. 3. Typical reinforcement of panel.
TESTING APPARATUS AND PROCEDURE

Each specimen was positioned vertically into the testing machine as shown in Fig. 7. It should be noted that the test specimens were rotated 90 degrees with respect to Figs. 2, 5 and 6 for testing convenience. 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, equivalent to 2 MPa and 4 MPa (290 psi and 580 psi), were applied normal to the joint. A post-tensioning scheme consisting of four hydraulic jacks, Dywidag bars and a series of plates designed to allow deformation of the joint was used to measure the applied preload as shown in Fig. 8. An additional post-tensioning scheme was
Fig. 5. Specimen configuration.

Fig. 6. Mechanical strain gage locations.

The testing procedure was started by applying the normal preload to the designed level, followed by an initial reading of all the instrumentation. The testing machine was used to apply the vertical shear load in increments of 100 kN (22.5 kips). At each increment, all
readings of the instrumentation were recorded, followed by marking of the cracks. The experiment was terminated after extensive deformation and significant drop of the load carrying capacity was observed. Descriptions of the tested specimens, including the material properties of the concrete and dry pack grout, are given in Table 1.

OVERALL BEHAVIOR

A summary of the test results and overall behavior of the specimens are discussed in this section. For detailed information, consult Ref. 10.

Connection Types and Behavior

Connection Type I (Dry Pack Grout Only) — Typical displacement of the joint parallel to the applied shear load for Specimen SP21 under a preload level of 4 MPa (580 psi) is shown in Fig. 9.
Table 1. Summary of variables considered in experimental program.

<table>
<thead>
<tr>
<th>Normal preload level (MPa)</th>
<th>Specimen mark</th>
<th>Type of connection</th>
<th>Concrete compressive strength $f_c$ (MPa)</th>
<th>Dry pack compressive strength $f'_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SPI 1</td>
<td>I</td>
<td>31.9</td>
<td>46.4</td>
</tr>
<tr>
<td></td>
<td>SPI 2</td>
<td>II</td>
<td>32.3</td>
<td>49.7</td>
</tr>
<tr>
<td></td>
<td>SPI 13</td>
<td>III</td>
<td>43.5</td>
<td>56.6</td>
</tr>
<tr>
<td></td>
<td>SPI 14</td>
<td>IV</td>
<td>46.5</td>
<td>66.5</td>
</tr>
<tr>
<td></td>
<td>SPI 12C</td>
<td>Precracked II</td>
<td>43.2</td>
<td>59.4</td>
</tr>
<tr>
<td></td>
<td>SPI 4</td>
<td>IV</td>
<td>46.5</td>
<td>66.5</td>
</tr>
<tr>
<td></td>
<td>SPI 2</td>
<td>II</td>
<td>41.5</td>
<td>56.6</td>
</tr>
<tr>
<td></td>
<td>SPI 21</td>
<td>I</td>
<td>34.5</td>
<td>67.6</td>
</tr>
<tr>
<td></td>
<td>SPI 22</td>
<td>II</td>
<td>49.3</td>
<td>56.9</td>
</tr>
<tr>
<td></td>
<td>SPI 23</td>
<td>III</td>
<td>52.8</td>
<td>60.2</td>
</tr>
<tr>
<td></td>
<td>SPI 24</td>
<td>IV</td>
<td>32.4</td>
<td>59.6</td>
</tr>
<tr>
<td></td>
<td>SPI 25</td>
<td>V</td>
<td>32.4</td>
<td>59.6</td>
</tr>
</tbody>
</table>

Note: 1 MPa = 145 psi.
9. The joint exhibited insignificant deformation prior to cracking of the grout. Substantial and sudden displacement, accompanied by significant loss of stiffness and the load carrying capacity, was observed at the initiation of cracking of the grout layer as shown in Fig. 10. The specimen exhibited a steady deterioration of the load carrying capacity due to smoothing of the interface surfaces with increased displacement. The resistance after cracking was proportionate to the level of the applied preload normal to the joint.

**Connection Type II (Dry Pack Grout and Continuity Bars) —** Similar to Joint Type I, the deformation was insignificant before cracking, as shown in Fig. 11 for Specimens SP12 and SP22 under the two predescribed preload levels. The presence of the continuity bars enhanced the resistance after cracking due to the effect of dowel action. The load carrying capacity steadily increased by
subjecting the joint to large deformation as illustrated in Fig. 11.

It should be noted that the higher cracking load of Specimen SP22 could be attributed to the significantly higher compressive strength of the dry pack grout used for this specimen. The behavior of the precracked specimen, SP12C, was identical to Specimen SP12 after cracking, as shown in Fig. 11. Typical failure of this type of connection is shown in Fig. 12 for Specimen SP12C.

**Connection Type III (Dry Pack, Continuity Bars and Shear Connectors Type A)** — Use of mechanical shear connectors with Anchorage Detail A in addition to the continuity bars and dry pack grout had no influence on the joint behavior before cracking of the grout, as observed for Specimens SP13 and SP23. However, the gradual reduction of the stiffness rather than the sudden drop of the load carrying capacity, shown in Fig. 13, was evidence of the additional contri-

---

Fig. 13. Load-displacement relationship of Connection Type III.

25.4 mm = 1 inch
1 kN = 0.225 kips

Fig. 14. Typical failure of Connection Type III.
Fig. 15. Load-displacement relationship of Connection Type IV.

The behavior before cracking of Specimens SPI4 and SP24 with Anchorage Type B was identical to that of specimens with Type A anchorage. However, both specimens with Type B anchorage exhibited a faster rate of stiffness reduction, as shown in Fig. 15. This behavior could be attributed to earlier bond failure of Anchorage Detail B in comparison to Detail A. Typical failure of this
type of connection is shown in Fig. 16 for Specimen SP24.

Connection Type V (Dry Pack, Shear Keys) — One specimen with dry pack multiple shear keys connection, SP25, was tested in this program. As illustrated in Fig. 17, the behavior indicates a gradual loss of stiffness due to the progressive formation of failure surfaces, which are due to the seating characteristics of the keys and continuous localized crushing of the concrete and grout as shown in Fig. 18. Continuous cracking of the grout results in gradual losses in the stiffness rather than the sudden drop of load resistance observed in other types of connections. As shown in Fig. 18, the ultimate resistance occurred as a result of the complete cracking of the concrete at the bases of the shear lugs. Using the shear keys connection increased the load carrying capacity by approximately 40 percent when compared to the plain surface
Behavior of the five different types of connections considered in this program, under a preload level of 4 MPa (580 psi), is shown in Fig. 19.

Ductility

Ductility of the various types of connections tested in this program was determined based on the total energy absorbed by the joint at a selected displacement of 5.4 mm (7/8 in.), achieved by all the tested specimens. Fig. 20 compares the measured ductility of the various joints to the ductility of the dry pack connection, Type I, at the two preload levels. Obviously, using mechanical shear connectors clearly enhanced the ductility by an average of 80 percent. It should also be noted that using shear keys connection improved the ductility by 65 percent.

Fig. 19. Behavior of all connections under a preload of 4 MPa (580 psi).

Fig. 20. Ductility ratio of the different connections.
Cracking Strength

Based on the measured experimental results, it was found that regardless of the type of mechanical shear connectors used, the cracking strength stress, \( v_{cr} \), depends on the tensile strength of the grout, \( f_t \), and the level of the applied preload, \( \sigma_n \), as follows:

\[
v_{cr} = \sqrt{f_t(\sigma_n + f_t)}
\]  

(1)

Accordingly, the cracking load of the connection, \( V_{cr} \), can be estimated based on the area of the concrete interface along the cracked surface, \( A_c \), as follows:

\[
V_{cr} = A_c v_{cr}
\]  

(2)

Using an estimated tensile strength of the grout, \( f_t \), in terms of the ultimate compressive strength, \( f'_c \), of 0.5 \( \sqrt{f_c} \), could lead to a very reliable prediction of the cracking load. This is shown in Fig. 21.

Ultimate Resistance

Current design sources\(^*\) were used to predict the ultimate resistance of the joints in this study, and the results were compared to the measured values as shown in Fig. 22. Using the well-known friction theory\(^*\) and including the weld area of the mechanical shear connector as part of the clamping action, the ultimate shear resistance of the connection, \( V_u \), on the contact surface \( A_c \) could be determined as follows:

Model 1:

\[
V_u = \mu(\sigma_n + \rho_b f_y + \rho_w f_{we}) A_c
\]  

(3)

where \( \rho_b \) is the percentage of steel of the continuity bars of yield strength \( f_y \), and \( \rho_w \) is the percentage of steel of the weld area of yield strength \( f_{we} \).

For an average coefficient of friction, \( \mu \), of 0.7 based on the measured ultimate load, the predicted ultimate shear resistance of the various connections
Fig. 22. Predicted and measured ultimate loads.

Fig. 23. Prediction capability of existing and proposed models.
tested in this program is compared to the measured values in Fig. 23. This model could be modified by considering the shear resistance of the weld, $V_w$, as an additive component to the shear friction as follows:

Model 2:

$$V_u = \mu (\sigma_u + \rho_b f_u) A_c + V_w$$

(4)

where $V_u$ is the shear strength of the weld, $A_c$, of the mechanical shear connector.$^9$

$$V_w = 0.5 A_w f_{wuc}$$

(5)

Predicted ultimate shear resistance using Model 2, Eq. (4), and a friction coefficient factor of 0.7 is compared to the measured values in Fig. 23. The proposed model introduced in this investigation is to evaluate the ultimate shear resistance of the joint as a summation of the contribution of each individual component. The components are the friction due to aggregate interlock at the joint, the shear resistance due to dowel action of the continuity bars, and the shear resistance of the weld as follows:

Proposed Model:

$$V_u = \mu \sigma_u A_c + A_b f_u / \sqrt{3} + V_w$$

(6)

where $A_b$ is the area of the continuity bars.

The predicted ultimate shear resistance based on the proposed model using a friction coefficient factor of 0.7 is also given in Fig. 23. The prediction reliability of this model appears to be superior to Models 1 and 2. The overestimation of the strength of Specimen SP24 could be attributed to the premature failure of Anchorage Detail B used for this connection.

**NUMERICAL EXAMPLE**

Consider a 200 mm (8 in.) precast concrete loadbearing shear wall panel at the fourth floor of a 26-story high rise apartment complex (see Fig. 24). The required problem is to determine the ultimate shear resistance of a typical shear connection.

To solve the problem, assume that the following information is given (see next page):
Equivalent dead load at the connection level = 360 kN/m (24.7 kips/ft).
Total horizontal shear load at the connection level = 800 kN (180 kips).
Yield strength of the continuity bars: $f_y = 400$ MPa (58 ksi).
Yield strength of the welding electrode: $f_{ye} = 480$ MPa (69.6 ksi).
Compressive strength of the concrete: $f_c = 40$ MPa (5 ksi).

It should be noted that the method described in this paper is based solely on the effect of shear loads. Therefore, one-half of the length of the panel will be considered in evaluating the shear resistance of the connection in order to account for the presence of bending moments.

Based on the specified dead load, the stress normal to the connection is:

$$
\sigma_n = \frac{360}{200} = 1.8 \text{ MPa (261 psi)}
$$

The area of the contact surface is:

$$
A_c = \left(\frac{8200}{2}\right) (200) = 820 \times 10^3 \text{mm}^2 (1271 \text{ in.}^2)
$$

Using an estimated tensile strength of the grout, $f_{tg}$ of 0.5 $f_{tg}$, the cracking strength of the connection can be estimated using Eq. (2):

$$
V_{cr} = [820 \times 10^3 \sqrt{2.96(1.8 + 2.96)}] 10^{-3}
= 3077 \text{ kN (692 kips)}
$$

Based on the proposed model, Eq. (6), the nominal ultimate shear resistance of the connection can be predicted:

$$
V_u = \left[ 0.7(1.8)(830 \times 10^3) + 6(500) \frac{400}{\sqrt{3}} + 0.5 \left( \frac{11}{2} \times 225 \right) (480) \right] 10^{-3}
= 2146 \text{ kN (483 kips)}
$$

The ultimate resistance is considerably higher than the factored load due to wind:

$$
V_f = 1.7(800) = 1360 \text{ kN (306 kips)}
$$

Since the cracking strength exceeds the ultimate shear resistance of the connection, the deformation of the connection will be insignificant.

**CONCLUSIONS**

The various limit states behavior of the connection typically used for precast concrete loadbearing shear wall panels is determined. Based on the test results of ten prototype specimens, it was found that regardless of the mechanical connectors used, the cracking strength depends on the tensile strength of the grout and the level of the preload applied normal to the joint.

A proposed mathematical model based on the individual contribution of each component of the connection is introduced. The predicted ultimate shear resistance of the connection was found to be in good agreement with the measured values.

The results of a pilot specimen tested with a shear keys connection indicated superior performance in comparison to the typical connections. Further research is currently in progress to investigate the behavior of the shear keys connection with dry pack grout.

**ACKNOWLEDGMENT**

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REFERENCES

1. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI318-83)," Detroit, Michigan, 1983.


APPENDIX — NOTATION

\( A_b = \) area of continuity bars

\( A_c = \) contact area of concrete interface subject to compressive stresses

\( A_w = \) throat area of weld

\( f'_c = \) compressive strength of concrete

\( f'_d = \) compressive strength of dry pack grout

\( f_t = \) tensile strength of dry pack grout

\( f_y = \) yield strength of continuity bars

\( f_{ye} = \) yield strength of weld

\( V_{cr} = \) cracking strength of connection

\( V_{cr} = \) cracking load of connection

\( V_u = \) ultimate shear resistance of connection

\( V_w = \) shear resistance of weld

\( \mu = \) coefficient of friction

\( \rho_b = \) percentage of steel of continuity bars

\( \rho_w = \) percentage of steel of weld area

\( \sigma_n = \) compressive stress normal of connection

* * *

NOTE: Discussion of this paper is invited. Please submit your comments to PCI Headquarters by October 1, 1989.