BEHAVIOUR AND DESIGN OF SHEAR CONNECTIONS
FOR LOAD BEARING WALL PANELS

by

H.R. Foerster, S.H. Rizkalla, and J.S. Heuvel

A total of five different connections for load bearing wall panels were subjected to monotonic shear load, up to failure. Two levels of preload, applied normal to the connection, were used. The different connection configurations included dry pack grout, continuity reinforcement steel bars, mechanical shear connectors with two different anchorage details and dry-packed shear keys.

Test results were used to evaluate the friction coefficient factor and to refine and calibrate a proposed rational mathematical model introduced to predict the shear carrying capacity before and after cracking.

Key Words: reinforced concrete, shear wall, connection, grout, cracking strength, strain, shear friction, dowel action, shear key tests, shear strength.
During the past decade, use of precast load bearing shear wall panels for highrise construction has become very popular due to the high quality control achieved at the precast manufacturing plants and ease of panel assembly at the erection site, Fig. 1. At the site, the precast panels are assembled and 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. Using this system eliminates the use of falsework, and minimizes the use of temporary supports.

The current design sources (1,2,3) provide very limited information to describe the behaviour and the ultimate shear carrying capacity of the connections described in this paper. The information available in the literature (4,5,6,7,8) is mainly applicable to describe 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 predescribed 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 behaviour 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 7/8 in.) and overall dimensions of 1660 mm (65 3/8 in.) and 1290 mm (50 3/4 in.), as shown in Fig. 2. The dimensions correspond to a prototype scale of the precast panels typically used for highrise 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 connection 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 (6 x 12 in.) concrete cylinders and 75 mm (3 in.) cubes, respectively. The concrete was supplied by a local ready-mix concrete company with a maximum aggregate size of 14 mm (1/2 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 described before. Specially designed temporary steel brackets were used for specimens with dry pack grout only. The dry pack grout 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 (3/4 in.) wide joint space between the two panels. The mixture was of a dry consistency which allowed for tamping and compaction of the material.

Electric resistance strain gauges 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 gauges 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.

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 (290 psi) and 4 MPa (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 were used to measure the applied preload as shown in Fig. 8. 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 (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. Description of the tested specimens, including the material properties of the concrete and dry pack grout, are given in Table 1.

OVERALL BEHAVIOUR

A summary of the test results and overall behaviour of the specimens are discussed in this section. For detailed information, the reader should consult Ref. (10).

Connection Type I (Dry Pack Grout only)

Typical displacement of the joint parallel to the applied shear load for specimen SP21 under preload level of 4 MPa (580 psi) is shown in Fig. 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 the 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 behaviour 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 Connector 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 behaviour before cracking of the grout as observed for specimen 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 contribution of the shear connectors. Behaviour of the specimen suggested also that the presence of the mechanical shear connectors enhanced the clamping action and the overall ductility of the joint as evidence by the large deformation before failure as shown in Fig. 14 for specimen SP13.

Connection Type IV (Dry Pack, Continuity Bars and Shear Connectors Type B)

The only difference between joint Type III and Type IV lies in the anchorage detail used for the shear connector as described in Fig. 4. The behaviour before cracking of specimens SP14 and SP24 with anchorage Type B were identical to the previous specimens with Type A anchorage. However,
both specimens exhibited a faster rate of stiffness reduction as shown in Fig. 15. This behaviour 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, SP25, with dry pack shear keys connection was tested in this program. The behaviour, illustrated in Fig. 17, indicates a gradual loss of the stiffness due to the progressive formation of failure surfaces 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 sudden drop of the load resistance observed in other types of connections.

The ultimate resistance occurred as a result of the complete cracking of the concrete at the bases of the shear lugs as shown in Fig. 18. Using the shear keys connection increased the load carrying capacity by approximately 40 percent as compared to the plain surface grouted joint. Further experimental work is currently in progress, to examine the behaviour of various shear keys configurations at different preload load levels normal to the joint. Behaviour of the five different types of connection 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 (7/32 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.

**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, \( Q_n \), as follows:

\[
v_{cr} = \sqrt{\frac{f_t}{Q_n + f_t}}
\]

(1)

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

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

(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 as shown in Fig. 21.

**Ultimate Resistance**

The current design sources (1,2,3) 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 known friction theory (4,5,6,7,8) 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:

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

(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 \( f_{yw} \).

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 tested in this program is compared to the measured values in Fig. 23.

The above 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:

\[ V_u = \mu (\sigma_n + \rho_b f_y) A_c + V_w \]  

(4)

where \( V_w \) is the shear strength of the weld, \( A_w \), of the mechanical shear connector (9).

\[ V_w = 0.5 A_w f_{yw} \]  

(5)

Predicted ultimate shear resistance using model 2, equation 4, and a friction coefficient factor of 0.7 are 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:

\[
V_u = \mu \sigma_n A_c + A_b \frac{f_y}{\sqrt{3}} + V_w
\]  \hspace{1cm} (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 using this model seems 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**

In order to determine the ultimate shear resistance of a typical shear connection, shown in Fig. 24, for a 200mm (8 in.) precast concrete load bearing shear wall panel at the fourth floor of a twenty-six storey high-rise apartment complex is determined, the following information is provided:

- The equivalent dead load at the connection level = 360 kN/m (24.7K/ft)
- The total horizontal shear load at the connection level = 800 kN (180 Kips)
- Yield strength of the continuity bars, \( f_y = 400 \text{ MPa (58 Ksi)} \)
- Yield strength of the welding electro rod, \( f_{yw} = 480 \text{ MPa (69.6 Ksi)} \)
- Compressive strength of the concrete, \( f'_c = 40 \text{ MPa (5 Ksi)} \)

It should be noted that the method described in this paper was 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:

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

Using an estimated tensile strength of the grout, \( f_t \), of \( 0.5 \sqrt{f_g} \), the cracking strength of the connection can be estimated using equation (2) as follows:

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

Based on the proposed model, equation (6), the nominal ultimate shear resistance of the connection, can be predicted as follows:

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

\[ = 2146 \text{ KN (483 Kips)} \]

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

\[ V_f = 1.7 \times 800 = 1360 \text{ KN (306 Kips)} \]

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 behaviour of the connection typically used for precast concrete load bearing 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 behaviour of the shear keys connection with dry pack grout.

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BIOPGRAPHICAL SKETCHES

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FIGURE LEGENDS

Figure 1  Precast Highrise Building
Figure 2  Panel Configuration
Figure 3  Typical Reinforcement of Panel
Figure 4  Connection Configurations
Figure 5  Specimen Configuration
Figure 6  Mechanical Strain Gauge Locations
Figure 7  Test Setup
Figure 8  Preload Apparatus
Figure 9  Typical Failure of Connection Type I
Figure 10  Load-Displacement Relationship of Connection Type I
Figure 11  Load-Displacement Relationship of Connection Type II
Figure 12  Typical Failure of Connection Type II
Figure 13  Load-Displacement Relationship of Connection Type III
Figure 14  Typical Failure of Connection Type III
Figure 15  Load-Displacement Relationship of Connection Type IV
Figure 16  Typical Failure of Connection Type IV
Figure 17  Load-Displacement Relationship of Connection Type V
Figure 18  Failure of Connection Type V
Figure 19  Behaviour of all Connections under a Preload of 4 MPa
Figure 20  Ductility Ratio of the Different Connections
Figure 21  Predicted and Measured Cracking Loads
Figure 22  Predicted and Measured Ultimate Loads

(new)
Figure 23  Prediction Capability of Existing and Proposed Models
Figure 24  Design Example
NOTATION SECTION

$A_b$ area of the continuity bars

$A_c$ contact concrete interface area subjected to compressive stresses

$A_w$ throat area of the weld

$f'_c$ compressive strength of the concrete

$f'_g$ compressive strength of the dry pack grout

$f_t$ tensile strength of the dry pack grout

$f_y$ yield strength of the continuity bars

$f_{yw}$ yield strength of the weld

$v_{cr}$ cracking strength of the connection

$V_{cr}$ cracking load of the connection

$V_u$ ultimate shear resistance of the connection

$V_w$ shear resistance of the weld

$\mu$ coefficient of friction

$\rho_b$ percentage of steel of the continuity bars

$\rho_w$ percentage of steel of the weld area

$\sigma_n$ compressive stress normal of the connection
TABLE 1  
OVERVIEW OF VARIABLES CONSIDERED  
IN THE 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, $f'_g$, (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>SP11</td>
<td>I</td>
<td>31.9</td>
<td>46.4</td>
</tr>
<tr>
<td></td>
<td>SP12</td>
<td>II</td>
<td>32.3</td>
<td>49.7</td>
</tr>
<tr>
<td></td>
<td>SP13</td>
<td>III</td>
<td>43.5</td>
<td>56.6</td>
</tr>
<tr>
<td></td>
<td>SP14</td>
<td>IV</td>
<td>46.5</td>
<td>66.5</td>
</tr>
<tr>
<td></td>
<td>SP12C</td>
<td>Precracked II</td>
<td>43.2</td>
<td>59.4</td>
</tr>
<tr>
<td>4</td>
<td>SP21</td>
<td>I</td>
<td>41.5</td>
<td>56.6</td>
</tr>
<tr>
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</tr>
<tr>
<td></td>
<td>SP25</td>
<td>V</td>
<td>32.4</td>
<td>59.6</td>
</tr>
</tbody>
</table>

1 MPa = 145 psi
Type I

20 mm wide grouted joint space typical

Type II

Continuity bars (25 M (No. 8) reinforcing welded to 75 x 75 x 10 (3 x 3 x 3/8) angle iron)

770 mm c/c (centered in the connection) typical

Type III

Shear connector type A
(S-section welded to angle iron with connection plate)

Type IV

Shear connector type B
(15 M (No. 5) reinforcing and plate with connection plate)

Type V

100 mm typical
35 mm typical

"Dry pack" grouted shear keys

4 Shear key lugs typical

25.4 mm = 1 inch
1 MPa = 145 psi

Fig (4)
25.4 mm = 1 inch
1 kN = 0.225 kips
Load (kN)

Displacement (mm)

1 kN = 0.225 kips
25.4 mm = 1 inch

Sp 23
Sp 13
25.4 mm = 1 inch
1 kN = 0.225 kips

SP 25
25.4 mm = 1 inch
1 kN = 0.225 kips
Fig (20)
Fig (21)
The diagram shows the relationship between predicted load (kN) and measured load (kN). The graph includes data points for ACI, PCI, and CPCI, each represented by different markers. The lines indicate trends for each category. The legend indicates that 1 kN = 0.225 kips.
WELD
\[ \text{II} \sqrt{225} \]

3-25 M

20 mm
DRY-PACK

3-25 M

\[ 8200 \text{ mm} \]

\[ A_s / 25 \text{ M} = 500 \text{ mm}^2 \]
25.4 mm = 1 inch

Fig (24)