Horizontal Post-Tensioned Connections for Precast Concrete Loadbearing Shear Wall Panels

This paper presents the results of nine prototype specimens tested to investigate the behavior and capacity of post-tensioned horizontal connections typically used for precast loadbearing shear wall panels subjected to monotonic shear loading. Shear walls supporting hollow-core floor slabs were included. Two 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. A numerical design example is included to show the application of the proposed mathematical model.

Precast loadbearing shear wall panels are used extensively for high rise building construction, due to the high quality control that can be achieved with off-site fabrication, the reduction of construction time and, consequently, reduction in cost. In addition, precast concrete construction is seldom interrupted by adverse weather conditions.

Precast concrete medium and high rise buildings normally consist of precast concrete loadbearing shear wall panels arranged in both the longitudinal and transverse directions of the building to resist lateral loads, as shown in Fig. 1. The major difference between the longitudinal and transverse shear walls is mainly the type of horizontal con-
Transverse shear walls are normally used to support the hollow-core slab of the floor system, whereas the longitudinal shear walls run parallel to the hollow-core slab and do not include the hollow-core slab at their connection, as shown in Fig. 2.

The connections between the panels are extremely important. A well designed simple connection normally minimizes construction time, requires minimal falsework and maintains the overall integrity of the structure.

A recent innovative technique, utilized in the construction of horizontal connections for loadbearing shear wall panels, is the use of vertical post-tensioning tendons. 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 packed with drypack. After the erection of several stories, the tendons are post-tensioned, the ducts are grouted and the temporary braces are removed.

Until now, the post-tensioned horizontal connection, with or without a hollow-core slab, has not been adequately examined so as to gain a thorough understanding of its shear behavior under the various limit states. Also, the available literature on the subject is not directly applicable to this type of connection.

**RESEARCH SIGNIFICANCE**

The primary objective of this research is to investigate the behavior of post-tensioned connections for precast concrete loadbearing shear wall panels subjected to monotonic shear loading conditions. Connections of shear wall panels supporting the hol-
low-core slab were included. Two load levels normal to the connection were considered to simulate the gravity and permanent loads. The research includes development of rational mathematical models proposed to predict the behavior and capacity of these connections at various limit states. The maximum shear capacity and the nominal shear strength based on the proposed mathematical models compared favorably with the measured values.

**BACKGROUND**

Horizontal connections are typically reinforced with a combination of continuity bars and mechanical shear connectors. In a previous study conducted at the University of Manitoba, it was found that the shear capacity of the horizontal connection may be predicted as the sum of the contributions from the friction resistance of the concrete interfaces, the dowel action of the continuity bars and the shear resistance of the mechanical shear connectors. To improve the shear capacity of this type of connection, some fabricators introduced the use of multiple shear keys along the horizontal portion of the joint surface of the panel. The presence of these shear keys has been shown to enhance the shear capacity in comparison to the plain surface connection.

Several tests have been conducted in Europe to determine the shear capacity of horizontal connections supporting a hollow-core slab. However, the details of the European connections differ from those used in North American practice, and their results are not directly applicable.

The results of two investigations related to the behavior of connections used in North American practice were significantly different and no effort was made to separate the contribution of the various components of the connections to the shear strength of the connections.

Fig. 3 shows a typical North American type horizontal connection supporting a hollow-core slab. The majority of tests conducted for this type of connection investigated the vertical load carrying capacity. It was found that the total applied vertical stress is distributed to the various components of the connection according to their relative stiffness, as shown in Fig. 4. The total vertical stress normal to the connection, \( \sigma_n \), is distributed to the bearing area of the hollow-core slab in contact with the drypack, Column 1, \( \sigma_{n1} \), and the bearing area of the concrete fill between the hollow-core slabs, Column 2, \( \sigma_{n2} \), using the following expressions:

\[
\sigma_{n1} = \frac{(2b_1 + b_2) \sigma_n}{(2b_1 + b_2k_2/k_1)} \\
\sigma_{n2} = \sigma_{n1} \left( \frac{k_2}{k_1} \right)
\]

where \( k_1 \) and \( k_2 \) are the equivalent spring constants of Columns 1 and 2, respectively. These constants can be determined using the thickness, \( t_i \), and the elastic modulus, \( E_i \), for the individual components of the connection as follows:

\[
k_1 = \frac{t}{\sum t_i / E_i} \quad i = 1, \ldots, 4
\]

\[
k_2 = \frac{t}{\sum t_i / E_i} \quad i = 4, 5
\]

where the different components are designated:

- \( i = 1 \) for bearing pad
- \( i = 2 \) for hollow-core slab
- \( i = 3 \) for concrete fill in the cores of the hollow-core slab
- \( i = 4 \) for drypack
- \( i = 5 \) for concrete fill between hollow-core slabs

**EXPERIMENTAL PROGRAM**

Nine full scale specimens were tested in this experimental program. The first two groups of specimens were designed to simulate the transverse interior shear wall connections supporting the hollow-core floor slabs. The third group was designed to simulate the horizontal connection of the longitudinal shear walls used for the elevator shafts and stairwells. For each category, the presence of post-tensioning was investigated and two levels of load normal to the connection were considered to simulate the effects of gravity and permanent loading conditions.

In this paper, the first digit of the specimen mark, given in Tables 1 and 2, represents the specimen number.
In this investigation:

\[ b_1 = b_2 = 50 \text{ mm} \]

\[ A_{bh} = L \times \Sigma b_1 = 2 \left[ 50 \times 1200 \right] = 120000 \text{ mm}^2 \]

\[ A_{bc} = L \times b_2 = 50 \times 1200 = 60000 \text{ mm}^2 \]

\[ A_{wh} = 2 \left[ 50 \times \Sigma \text{web} \right] = 2 \left[ 50 \times (5\times40 + 2\times50) \right] = 30000 \text{ mm}^2 \]

Note: 1 mm = 0.039 in.

---

**Fig. 4.** Distribution of vertical load and dimensions of typical horizontal connection with hollow-core slab.

(Note: 1 mm = 0.039 in.)

**Table 1.** Material properties.

<table>
<thead>
<tr>
<th>Specimen mark</th>
<th>Cylinder compressive strength</th>
<th>Tensile strength of concrete fill (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Wall panel concrete (MPa)</td>
<td>Concrete fill (MPa)</td>
</tr>
<tr>
<td>1HD4</td>
<td>40.90</td>
<td>55.70</td>
</tr>
<tr>
<td>2HD8</td>
<td>41.30</td>
<td>45.10</td>
</tr>
<tr>
<td>3HD8</td>
<td>39.70</td>
<td>56.10</td>
</tr>
<tr>
<td>4HP4</td>
<td>34.10</td>
<td>52.30</td>
</tr>
<tr>
<td>5HP8</td>
<td>38.80</td>
<td>41.10</td>
</tr>
<tr>
<td>6HP8</td>
<td>37.60</td>
<td>53.30</td>
</tr>
<tr>
<td>7PD4</td>
<td>38.10</td>
<td>9.70</td>
</tr>
<tr>
<td>8PD8</td>
<td>46.20</td>
<td>9.70</td>
</tr>
<tr>
<td>9PD8</td>
<td>37.80</td>
<td>11.50</td>
</tr>
</tbody>
</table>

The following two characters indicate the particular combination of parameters as follows:

HD: hollow-core slab and drypack only;
HP: hollow-core slab and post-tensioning; and
PD: post-tensioned with drypack only

Note that the last digit of the mark, given in Tables 1 and 2, represents the level of stress [4 and 8 MPa (580 and 1160 psi)] normal to the connection.

A typical specimen with hollow-core slab and post-tensioning is shown in Fig. 5. 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 slab and the top panel is filled with drypack. Load was applied normal to the connection to simulate the effects of gravity, and the shear load was applied directly through the center of the connection, as shown in Fig. 5.

To produce equivalent pressure on the bearing pad as attained by the long
span of hollow-core units used in an actual structure, a loading apparatus consisting of threaded bars and load cells was used to apply equivalent pressure on the Korolath bearing pads. Only 150 mm (6 in.) portions of the hollow-core slabs were used on each side of the connection to model the actual connections since this portion of the cores of the hollow-core slab is typically filled with concrete.

The strength of the concrete fill was comparable to that of the precast panels and the hollow-core slab, and is much stronger than the drypack.

Seven days after drypackaging, the strands were post-tensioned and the ducts were grouted using an expansive grout. Twenty-eight days later, the specimen was moved to the testing machine where the horizontal connection was tested in a vertical orientation, as shown in Fig. 6.

The edges of the panels were post-tensioned with a system of Dywidag bars to prevent premature cracking of the panels. The load normal to the connection was applied through a system of rollers to allow relative displacement of the two panels in the direction of the applied shear load.

Vertical and horizontal relative displacements from panel to panel and across the various interfaces were measured as shown schematically in Fig. 7. Electronic LVDTs were used on one surface of the panel while mechanical dial gauges were used on the other.

The measured material properties for the concrete used in the shear wall panels, concrete fill and drypack are listed in Table 1. The tensile strength of the concrete fill, based on the measured compressive strength, is also listed in Table 1. The hollow-core slab was supplied by Con-Force Structures Limited, Winnipeg, Manitoba, Canada, with a design strength of 52 MPa (7540 psi) and tensile strength of 4.3 MPa (627 psi).

**TEST RESULTS AND DISCUSSION**

The measured maximum and nominal shear strength of all the tested connections are listed in Table 2. The maximum shear capacity corresponds to the maximum load recorded during the test. The nominal shear strength corresponds to the load maintained after large slip deformation greater than 5 mm (0.2 in.).

The predominant mode of failure was due to slip along the drypack to panel interface, with the exception of one specimen in which slip occurred between the concrete fill and bearing pad to panel interface.

For Specimen 1HD4, without post-tensioning and subjected to stresses of 4 MPa (580 psi) normal to the connection, the ultimate shear capacity of the connection was maintained without any damage to the hollow-core slab. For Specimen 4HP4, which is identical to Specimen 1HD4 except for an additional 1.2 MPa (174 psi) due to post-tensioning stresses, some minor cracking of the hollow-core slab at the bottom of the connection was observed at failure. This cracking
could be due to the incomplete concrete filling of the bottom core of the hollow-core slab, since the cracks did not propagate along the connection with the increase of the applied slip deformation.

For the specimens tested under higher levels of stress normal to the connection, Specimens 2HD8, 3HD8, 5HP8 and 6HP8, extensive cracking of the hollow-core slab was observed at failure of the connection, as shown in Fig. 8. The cracking began prior to the maximum applied shear load, and the cracks propagated by increasing the applied slip deformation.

For Specimen 5HP8, a post-tensioned hollow-core specimen tested at the higher load level of 8 MPa (1160 psi) normal to the connection, it was found that damage occurred in the concrete wall panel, as shown in Fig. 9. Crushing and spalling of the concrete in this zone is attributed to the eccentricity of the applied shear load due to the presence of the hollow-core slab which induces a higher level of compressive stress in this vicinity.

The effect of the different variables investigated in this program on the behavior of the connection is summarized in the following sections.

(a) Effect of Load Normal to the Connection

The effect of load normal to the connection on the shear resistance of the connection is shown in Fig. 10. The two drypacked post-tensioned specimens, 8PD8 and 7PD4, were subjected to 8 and 4 MPa (1160 and 580 psi) stresses normal to the connection, respectively, in addition to the 1.2 MPa (174 psi) due to post-tensioning stresses. The behavior indicates that the increase in shear capacity is directly proportional to the increase of the load normal to the connection. This behavior is consistent with the friction theory presented in a previous study.¹

(b) Effect of Post-Tensioning

To demonstrate the effect of post-tensioning, Fig. 11 compares two hollow-core slab specimens, 4HP4 and 1HD4, with and without post-tensioning, respectively, subjected to 4 MPa (580 psi) stresses normal to the connection. The results indicate that the increase in the maximum and ultimate shear capacities are proportional to the increase in the load normal to the connection due to post-tensioning stresses. These results suggest that the

![Fig. 8. Typical cracking of hollow-core slab at failure.](image1)

![Fig. 9. Cracking of panel at failure.](image2)

![Fig. 10. Effect of load normal to connection.](image3)
post-tensioning enhances the friction resistance of the connection and may be accounted for by simply adding the post-tensioning equivalent stresses to the gravity load.

(c) Effect of the Presence of a Hollow-Core Slab

At the low stress level, 4 MPa (580 psi) normal to the connection, the presence of the hollow-core slab had no effect on the behavior or the capacity of the post-tensioned connections. This is clearly evident in Fig. 12, which compares load vs. slip curves for two post-tensioned specimens, Specimens 4HP4 and 7PD4, with and without a hollow-core slab.

However, at higher levels of load normal to the connection, the presence of the hollow-core slab significantly affects the behavior and the capacity of the connection. Fig. 13a illustrates the behavior of four specimens subjected to 8 MPa (1160 psi) load normal to the connection. The two specimens which support the hollow-core slab, 5HP8 and 6HP8, exhibit a significant reduction in the maximum and nominal shear strengths in comparison to Specimens 8PD8 and 9PD8 with drypack only. Also evident from Fig. 13a is the reduction in the ductility for the specimens supporting the hollow-core slab. The stiffness of the connections supporting the hollow-core slab was also reduced at high levels of load normal to the connection as illustrated in Fig. 13b.

Based on the observed behavior after cracking, the failure mechanism in horizontal connections with the 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. These two possible failure mechanisms are illustrated by the schematic load-slip Curves A and B in Fig. 14.

Curve A represents the typical load-slip behavior of a connection of a shear wall supporting a hollow-core slab and subjected to a high level of load normal to the connection. In this case, the maximum shear capacity, $V_h$, and the nominal shear strength, $V_{hn}$, of the connection are controlled by the cracking and nominal strength of the hollow-core slab. The behavior of the connection supporting the hollow-core slab subjected to low levels of load normal to the connection, Curve B, is mainly controlled by the friction model, $V_f$.

**PROPOSED MATHEMATICAL MODEL**

(a) Maximum Shear Capacity

Based on this investigation, the following mathematical model is proposed to predict the maximum shear capacity of post-tensioned horizontal connections with or without a hollow-core slab.

As shown in Fig. 14, the maximum shear capacity could be predicted as the lesser of that determined by the friction model, $V_f$, given by Eq. (2a), and the cracking capacity of the hollow-core slab, $V_h$, given by Eq. (2b):

$$V_f = \mu \sigma_c A_c$$  \hspace{1cm} (2a)

$$V_h = \frac{1}{2} (A_{sh} F_{t1} + A_{hc} F_{t2})$$  \hspace{1cm} (2b)
Fig. 13a. Effect of the presence of hollow-core slab at high stress level normal to the connection. Reduction in ductility.

Fig. 13b. Effect of the presence of hollow-core slab at high stress level normal to the connection. Reduction in stiffness.
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 drypack to panel interface. A coefficient of friction of 0.7 is proposed based on the test results of this investigation and the previous studies conducted at the University of Manitoba.

The maximum shear capacity based on the cracking capacity of the hollow-core slab, $V_{hc}$, can be predicted using the areas of the interface of the hollow-core slab at the connection, Column 1, $A_{bc}$, and the concrete fill, Column 2, $A_{bc}$, as described in Fig. 4. The magnified tensile strengths of the concrete of Column 1 and Column 2, $F_{t1}$ and $F_{t2}$, respectively, are based on the lowest tensile strength of the component materials in each column, $f_{t1}$ and $f_{t2}$, and the compressive normal stresses distributed to each column, $\sigma_{n1}$ and $\sigma_{n2}$, including the effect of prestressing.

Fig. 15 compares the maximum shear capacity of all nine specimens tested in this program and the strength as predicted by the proposed model. The predicted maximum shear capacities are in good agreement with the experimental results.

(b) Nominal Shear Strength

At failure, the normal and shear strength of the connection are resisted only by the area of the web of the hollow-core slab. This assumption is based on the observed complete loss of bond between the concrete fill and the hollow-core slab at failure. In some cases, the loss of bond may be caused by incomplete filling of the cores with the concrete fill during construction, the effects of shrinkage, or the propagation of cracks through the hollow-core slab and along the surface of the concrete fill in contact with the cores, as was observed in several of the specimens tested.

Based on a complete loss of bond, and therefore the prediction could be quite conservative, the nominal shear strength of horizontal connections with a hollow-core slab could be predicted as the lesser of that determined by the friction model, $V_f$, Eq. (2a), and the nominal shear strength of the hollow-core slab, $V_{hc}$, given by Eq. (3):

$$V_{hn} = \frac{1}{3} (A_{wh}F_{m} + A_{bc}F_{12})$$

where

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

$$F_{12} = \sqrt{f_{t2}(f_{t2} + \sigma_{n2})}$$

in which $A_{wh}$ is the summation of the area of the webs based on the thick-
maximum shear capacity

Fig. 15. Relationship between the measured and predicted maximum shear capacity of the connection.

nominal shear strength

Fig. 16. Relationship between the measured and predicted nominal shear strength of the connection.

November-December 1991

ness at mid-height of the hollow-core slab beneath the contact surface area \( A_{bh} \). For the given geometry of the hollow-core slab tested in this investigation, \( A_{bh} = 0.25A_{bh} \).

In this model, the contribution from the middle concrete fill of the connection, Column 2, remains unchanged while the contribution at the hollow-core slab is modified to reflect the loss of bond. The area of the hollow-core slab in contact with the drypack, \( A_{bh} \), is reduced by a factor of four; however, the normal stress \( \sigma_{ni} \) is also increased, by a factor of four, due to the reduction of the contact area at failure.

Fig. 16 compares the measured and the predicted nominal shear strength of all nine specimens tested in this program. The comparison suggests, as expected, that the proposed model provides a conservative lower bound for the nominal shear strength of horizontal connections with the hollow-core slab.

SAMPLE CALCULATION

Consider a typical post-tensioned connection for a precast concrete load-bearing interior shear wall panel at the base of a 32-story high rise building as shown in Fig. 4. To predict the maximum shear capacity and nominal shear strength of the connection, assume the following information is given:

Equivalent gravity and permanent load at connection level:
\[ = 8 \text{ MPa (1.16 ksi)} \]

Equivalent post-tensioning stresses:
\[ = 1.2 \text{ MPa (0.17 ksi)} \]

Therefore, total load normal to the connection,
\[ \sigma_n = 8 + 1.2 = 9.2 \text{ MPa (1.3 ksi)}. \]

The compressive strength of the various materials used for this connection, as described in Fig. 3, are:

Hollow-core slab:
\[ f'_{c2} = 52 \text{ MPa (7.54 ksi)} \]

Concrete fill in cores of hollow-core slab:
\[ f'_{c3} = 53.3 \text{ MPa (7.7 ksi)} \]

Drypack:
\[ f'_{c4} = 12.7 \text{ MPa (1.85 ksi)} \]
Concrete fill between hollow-core slab:

\[ f'_{c5} = 53.3 \text{ MPa (7.7 ksi)} \]

Based on the given compressive strengths, tensile strengths of the various materials estimated as \( f_t = 0.6 \sqrt{f'_c} \) (MPa) are:

Hollow-core slab:

\( f_{t2} = 4.33 \text{ MPa (0.63 ksi)} \)

Concrete fill:

\( f_{t5} = 4.38 \text{ MPa (0.635 ksi)} \)

Based on code equation \( E = 5000 \sqrt{f'_c} \) (MPa), the elastic modulus of the various components can be estimated as follows:

Hollow-core slab:

\[ E_2 = 36,054 \text{ MPa (5230 ksi)} \]

Concrete fill:

\[ E_5 = 36,503 \text{ MPa (5294 ksi)} \]

Drypack:

\[ E_4 = 17,820 \text{ MPa (2585 ksi)} \]

Assume elastic modulus of bearing pad:

\[ E_1 = 345 \text{ MPa (50 ksi)} \]

Based on the connection detailed configuration shown in Fig. 4, the following parameters are given:

\[ t_1 = 3 \text{ mm}, t_2 = 50 \text{ mm}, t_3 = 150 \text{ mm}, \]
\[ t_4 = 20 \text{ mm}, t_5 = 200 \text{ mm}, t = 203 \text{ mm}, \]
\[ A_{bh} = 120,000 \text{ mm}^2, A_{bc} = 60,000 \text{ mm}^2, \]
\[ A_{wh} = 30,000 \text{ mm}^2, A_c = 180,000 \text{ mm}^2, \]
\[ b_1 = 50 \text{ mm}, b_2 = 50 \text{ mm} \]

(1 mm = 0.0394 in.)

(a) Distribution of Normal Load

The equivalent stiffness of Columns 1 and 2, shown in Fig. 4, could be determined as follows:

\[ k_1 = \frac{t}{\frac{t_1}{E_1} + \frac{t_2}{E_2} + \frac{t_3}{E_3} + \frac{t_4}{E_4}} \]

\[ = \frac{203}{\frac{3}{345} + \frac{50}{36055} + \frac{150}{36503} + \frac{20}{17820}} \]

\[ = 13255 \text{ MPa (1922 ksi)} \]

(b) Maximum Shear Capacity

Based on the friction resistance given in Eq. (2a) and friction coefficient of 0.7:

\[ V_f = \mu \sigma_c A_c \]

\[ = 0.7 \times 9.2 \times 180,000 \]

\[ = 1160 \text{ MPa (260 ksi)} \]

To evaluate the capacity based on the hollow-core slab, the modified tensile strength can be evaluated as follows:

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

\[ = \sqrt{4.33(4.33 + 6.4)} \]

\[ = 6.81 \text{ MPa (2.15 ksi)} \]

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

\[ = \sqrt{4.38(4.38 + 14.85)} \]

\[ = 9.17 \text{ MPa (1.33 ksi)} \]

The maximum shear capacity, \( V_h \), can be determined using Eq. (2b):

\[ V_h = \% (A_{bh} F_{t1} + A_{bc} F_{t2}) \]

\[ = \% (120,000 \times 6.81 + 60,000 \times 9.17) \]

\[ = 912 \text{ kN (205 kips)} \]

Based on the lesser of \( V_f \) and \( V_h \), the maximum shear capacity is equal to 912 kN (205 kips).

For Specimen 6HP8 with the same prescribed material properties and detailed configuration, the measured maximum shear strength is 947 kN (212 kips), which is 3 percent higher than the predicted value.

(c) Nominal Shear Strength

The modified tensile strength using Eq. (3) can be estimated as:

\[ F_{m} = \sqrt{f_{t1}(f_{t1} + \sigma_{n1})} \]

\[ = \sqrt{4.33(4.33 + 6.4)} \]

\[ = 11.38 \text{ MPa (1.65 ksi)} \]

Using Eq. (3):

\[ V_{hn} = \% (A_{wh} F_{m} + A_{bc} F_{t2}) \]

\[ = \% (30,000 \times 11.38 + 60,000 \times 9.17) \]

\[ = 595 \text{ kN (134 kips)} \]

Based on the lesser of \( V_f \) and \( V_{hn} \), the nominal shear strength is equal to 595 kN (134 kips).

For Specimen 6HP8 with the same properties, the measured nominal shear strength is 880 kN (152 kips), which is 47 percent higher than the predicted value.

SUMMARY AND CONCLUSIONS

Nine prototype horizontal connections used for precast loadbearing shear wall panels were tested. The connections included the hollow-core slab with and without post-tensioning and a plain surface connection with post-tensioning.

The connections were subjected to monotonic shear loading conditions in order to investigate the various limit states behavior of
horizontal post-tensioned connections. The effects of load normal to the connections, post-tensioning and the presence of the hollow-core slab were investigated.

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

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

2. The effect of post-tensioning may be accounted for by adding the applied post-tensioning stresses to the gravity load normal to the connection.

3. For connections supporting the hollow-core slab, the failure mechanism could be controlled by friction resistance or by the shear capacity of the hollow-core slab.

4. For connections supporting the hollow-core slab at high levels of load normal to the connection, the stiffness and ductility of the connection are reduced in comparison to the same type of connection without the hollow-core slab. Shear failure in the hollow-core slab results in rapid degradation of the shear strength of the connection.

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

6. The nominal shear strength of the connection with the hollow-core slab is based on complete loss of bond between the concrete fill and the hollow-core slab.

7. The analytical model proposed for the maximum shear capacity of the connection is in excellent agreement with the measured values. However, the model proposed for the nominal shear strength provides a conservative lower bound.

ACKNOWLEDGMENT

This study was performed in the department of civil engineering at the University of Manitoba and was partly financed by the Precast/Prestressed Concrete Institute through a research fellowship awarded to Mrs. R. L. Hutchinson. Financial support of the experimental program was provided by the Natural Science and Engineering Research Council of Canada and Con-Force Structures Limited. The help of laboratory technicians Messrs. E. Lemke and M. McVey, and graduate students Messrs. Dong X., K. A. Soudki and J. S. West is greatly appreciated.

REFERENCES


APPENDIX — NOTATION

\( A_c \) = drypack interface surface area, mm\(^2\)

\( A_{bh} \) = total bearing area at the hollow-core slab interface with the drypack, Column 1, mm\(^2\)

\( A_{bc} \) = bearing area of the concrete fill portion of connection interface with the drypack, Column 2, mm\(^2\)

\( A_{wh} \) = summation of the cross-sectional area of the webs for the hollow-core slab within the connection, based on the thickness at mid-height. \( A_{wh} = b_1 \times \Sigma \) (web thicknesses), mm\(^2\)

\( b_1 \) = bearing width of the hollow-core slab in contact with the drypack, Column 1, mm

\( b_2 \) = bearing width of the concrete fill portion of connection in contact with the drypack, Column 2, mm

\( E_i \) = modulus of elasticity of connection component \( i \), MPa

\( f'_{ci} \) = compressive strength of material \( i \) at time of testing, MPa

\( f_{ti} \) = tensile strength of material \( i \), MPa

\( f_{r1} \) = lowest tensile strength of materials in Column 1

\( f_{r2} \) = lowest tensile strength of materials in Column 2

\( F_{r1} \) = magnified tensile strength of the material in Column 1 due to applied load normal to the connection, MPa

\( F_{r2} \) = magnified tensile strength of the material in Column 2 due to applied load normal to the connection, MPa

\( F_{in} \) = magnified tensile strength of the material in Column 1 at failure due to applied load normal to the connection, MPa

\( k_1 \) = equivalent spring constant of Column 1

\( k_2 \) = equivalent spring constant of Column 2

\( L \) = length of the connection, mm

\( t \) = thickness of connection including bearing pads, drypack and hollow-core slab or concrete fill, mm

\( t_i \) = thickness of component \( i \) of connection

\( V \) = applied shear load, \( N \)

\( V_f \) = frictional resistance, \( N \)

\( V_h \) = maximum shear capacity of the connection based on the cracking capacity of the hollow-core slab, \( N \)

\( V_{hn} \) = nominal shear strength of the connection based on the nominal shear strength of the hollow-core slab, \( N \)

\( \mu \) = coefficient of friction

\( \sigma_n \) = load normal to the connection applied to the wall panel, MPa

\( \sigma_{n1} \) = applied load normal to the connection distributed to Column 1 (hollow-core slab portion of the connection), MPa

\( \sigma_{n2} \) = applied load normal to the connection distributed to Column 2 (concrete fill portion of the connection), MPa

\( i = 1 \) for bearing pad

\( i = 2 \) for hollow-core slab

\( i = 3 \) for concrete fill in the hollow-core slab

\( i = 4 \) for drypack

\( i = 5 \) for concrete fill in the middle of the connection, between the hollow-core slabs