PERFORMANCE OF DOWEL TYPE PANEL-TO-FOUNDATION CONNECTIONS IN SLENDER CONCRETE PRECAST PANELS

L.S. Hogan¹, R.S. Henry², J. Hashemi³, and J.M. Ingham⁴

ABSTRACT

Precast concrete wall buildings represent a significant portion of the New Zealand building stock, particularly for low-rise industrial and commercial buildings. These buildings typically are constructed with tall, slender panels that have minimum reinforcement content, and are connected to the foundation with horizontal starter bars which are cast into the foundation following panel erection. Despite the prolific nature of this construction type, there is limited evidence of the seismic performance of existing connections between the panels and other structural elements. In addition, previous earthquakes, including the Canterbury and Kaikoura events, have revealed that these connections are often the most vulnerable aspect of this construction type. In response to this lack of evidence, an experimental program investigating the seismic response of panel to foundation connections was performed. The testing program focused on dowel type connections typically used for low-rise industrial or commercial buildings. The testing program consisted of over thirty concrete panels with a single layer of reinforcement, which incorporated both details currently used in practice as well as alternative connection details that have been proposed to improve robustness in connection performance, while still maintaining construction feasibility. Specimens were subjected to out-of-plane, in-plane, and bi-directional actions in order to assess the connection performance during different loading actions. A summary of this testing program is described herein, including the performance of both the existing and alternative details and recommendations for the design of these connections in new structures.

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Precast concrete wall buildings represent a significant portion of the New Zealand building stock, particularly for low-rise industrial and commercial buildings. These buildings typically are constructed with tall, slender panels that have minimum reinforcement content, and are connected to the foundation with horizontal starter bars which are cast into the foundation following panel erection. Despite the prolific nature of this construction type, there is limited evidence of the seismic performance of existing connections between the panels and other structural elements. In addition, previous earthquakes, including the Canterbury and Kaikoura events, have revealed that these connections are often the most vulnerable aspect of this construction type. In response to this lack of evidence, an experimental program investigating the seismic response of panel to foundation connections was performed. The testing program focused on dowel type connections typically used for low-rise industrial or commercial buildings. The testing program consisted of over thirty concrete panels with a single layer of reinforcement, which incorporated both details currently used in practice as well as alternative connection details that have been proposed to improve robustness in connection performance, while still maintaining construction feasibility. Specimens were subjected to out-of-plane, in-plane, and bi-directional actions in order to assess the connection performance during different loading actions. A summary of this testing program is described herein, including the performance of both the existing and alternative details and recommendations for the design of these connections in new structures.

Introduction

Precast concrete panels are a common construction form for low-rise industrial and commercial buildings in New Zealand. These buildings are typically a single storey with steel portal frames in the transverse direction, slender precast concrete panels on the exterior, and flexible steel diaphragms. Plan dimensions range from 35 to 60 m in the transverse direction and 60 to 100 m in the longitudinal direction. The precast concrete panels on the exterior of the building are typically 7 to 10 m tall and 150 mm to 200 mm thick, with a single layer of reinforcement. Despite these buildings being typically designed for an elastic or nominally ductile seismic response, they have been found to perform poorly in earthquakes, often due to failure of connections [1, 2]. While earthquake reconnaissance following the 2010/2011 Canterbury earthquake sequence in New Zealand found that overall, precast and tilt-up concrete buildings performed adequately [3], vulnerabilities were identified in the out-of-plane response of dowel connections.

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type panel-to-foundation connections for panels with a single layer of vertical reinforcement, particularly for panels that utilized threaded inserts embedded in the panels to connect starter bars to the foundation [4]. An example of a dowel type panel-to-foundation connection is shown in Figure 1. The use of threaded inserts has become popular in New Zealand for precast concrete panel construction because the starter bars can be screwed into the panels after they are erected and thus avoid the need to bend bars for transport and storage, thereby reducing labour and time on site prior to pouring of the foundation [5]. Because of the low out-of-plane strength of the panels, the demands on the starter bar reinforcement are typically below yield, and the pull-out strength of the inserts often governs the design of the foundation connection. As such, when the panel is loaded with a joint-opening moment, the concrete behind the insert head is required to act in tension to complete the primary load path (Figure 2), and when subjected to large moments, a vertical crack in the panel could develop as the concrete behind the insert head ruptures, causing a significant loss in strength and stiffness. To investigate the strength and deformation capacity of typical threaded insert connections, a series of precast panel-to-foundation connections of different nominal panel and joint connection strengths were subjected to out-of-plane loading, in-plane loading, and bi-directional loading.

![Figure 1: Schematic representation of a dowel type precast panel to foundation connection utilizing threaded inserts.](image1.png)

![Figure 2: Strut and tie representation of load path for panel-to-foundation connection using threaded inserts with a shallow embedment depth](image2.png)

**Out-of-Plane Performance**

**Test Set Up**

In order to simulate the out-of-plane demands on the foundation connection, a self-reacting test rig was constructed as is shown in Figure 3. The moment demand on the base of the wall was approximated using a 2.5 m cantilever because the moment demand and gradient of this set up was similar to that of the 10 m tall fixed base, pinned top prototype wall subjected to uniform face load. As these panels are only designed to support their own self-weight, no additional axial load was applied. Cyclic loading was applied at the wall top based upon ACI loading protocols [6, 7] to a maximum drift of 6.75%, at which point the hydraulic jack reached its stroke limit.
Existing Threaded Inserts Connection Details

Thirteen panel-foundation connections, which represented current construction practice in New Zealand, were tested in the out-of-plane direction to investigate the ability for the joint to develop the nominal panel strength as well as the robustness of the panel-to-foundation connection. Test panels were 150 mm thick, 2.5 m tall, and 900 mm wide with 40 MPa concrete, and had a single layer of reinforcement of either HD12 (12 mm diameter bars with $f_y = 500$ MPa) or HD16 (16 mm diameter bars with $f_y = 500$ MPa) longitudinal bars spaced at 270 mm. Two rows of starter bars were used and the starter bar dimensions were the same as the longitudinal reinforcement as shown in Figure 4. Insert spacing and depth was varied to be consistent with the range of current design and construction practices.

All panels performed in a similar manner with flexural cracking initiating at the foundation level and then propagating down behind the insert head as is shown in Figure 5. The panels with HD12 vertical reinforcement and starter bars were able to develop the nominal moment capacity in both the joint-opening and joint-closing directions, but degraded in strength and stiffness after a drift level of 2% as is shown in Figure 6. None of the panels with HD16 vertical reinforcement and starter bars were able to develop the nominal moment capacity in the joint-opening direction, and failed at a similar moment capacity to the HD12 panels. This finding was consistent with the failure mode being controlled by the tensile capacity of the concrete behind the insert heads. All panels exhibited a significantly pinched hysteretic response which resulted from a combination of the single layer of reinforcement and the low axial load on the panel. As the reinforcement elongated at a given drift level, the absence of significant axial load to assist with crack closure meant that a rotation that was equal to or greater than the previous drift level was required to close the flexural crack. Until this drift level was reached, the section behaved essentially as a pin. More details on the out-of-plane performance of existing panel-to-foundation details can be found in [8, 9].
Alternative Threaded Inserts Connection Details

In an effort to improve the performance of threaded insert panel-to-foundation connections, several alternative details were proposed and tested in the out-of-plane direction. These alternative details were developed to ensure that joint damage did not occur due to breakout of concrete behind the insert, and that the panel nominal moment capacity was developed. Seven panels were tested with three main categories of alternative details which included: increasing embedment depth, providing extra reinforcement in the joint region to force damage further up the panel height, and providing confinement in the joint region.

More details of the performance of alternative connection details can be found in [8], but each connection type performed similarly to that shown in Figure 7 and Figure 8. Damage was restricted to flexural cracking of the panel only, and the joint was undamaged and experienced limited rotations. Hysteretic responses of these panels were similar to that shown in Figure 9, with an overall stable hysteresis, albeit with significant pinching, in which the nominal moment capacity of the panel was developed in both the joint-opening and joint-closing directions.
In-Plane and Bi-Directional Performance

In-Plane and Bi-Directional Test Set Ups

With the out-of-plane behavior of several common and alternative panel-to-foundation connections established, the behavior of these connections was investigated for both in-plane and bi-axial loading to determine the overall seismic performance of panels with this dowel type connection. While these panels are utilized for cladding in their out-of-plane direction, they are relied upon as the main structural system in the longitudinal direction of the buildings in which they are constructed. As such, there was an interest in the effectiveness of the dowel connection to resist in-plane loads without spalling of the foundation or significant deformation of the starter bars. Additionally, the ability for the panels to resist in-plane loads after damage in the out-of-plane direction had initiated was investigated using bi-directional loading.

In-plane and bi-axial testing were performed on two types of threaded insert connections. The first connection utilized shallow embedment and was constructed as per current practice with 50 mm cover behind the insert. This connection detail is similar to that shown in Figure 4, except that the 8 mm thick nail plate at the panel-foundation interface in Figure 4 was not used. The second connection type was an alternative detail in which the insert head was pushed to the back of the panel, and is shown in Figure 7. Each panel was 150 mm thick and had a single layer of HD12 bars at 270 mm spacing, which corresponded to the minimum longitudinal reinforcement requirements and was representative of current construction practices for this building type. Transverse reinforcement consisted of HD10 bars (10 mm diameter bars with $f_y = 500$ MPa) at 200 mm spacing. Starter bars were aligned with the longitudinal reinforcement and were placed in two rows, 280 mm apart as is shown in Figure 10 and Figure 11.

The panels were 2 m wide and had a shear span ratio of 2 and an axial load ratio ($P/(A_{gf}f'_c)$) of 0.044% was applied to the wall to simulate the self-weight of a 10 m tall panel. The in-plane panels were tested as cantilever walls as is shown in Figure 12 and were subjected to cyclic loading based upon ACI loading protocols [6, 7]. The panels that were subjected to bi-directional
loading were tested using the Multi-Axis Substructure Testing (MAST) system at Swinburne University of Technology and were approximately 1.0 m shorter to enable them to fit under the MAST crosshead (Figure 11 and Figure 13). An in-plane moment was applied with respect to in-plane force to maintain the same shear span ratio of 2. The bi-directional loaded panels were tested with a drift ratio of 3:1 for the out-of-plane to in-plane drift demands, with the out-of-plane action occurring first in the loading protocol, to account for the increased flexibility of the structural system in the out-of-plane direction. Rotation about the vertical axis of the panel was restrained and the panel was allowed to rotate freely in the out-of-plane direction.

Figure 10: Reinforcement layout of bi-directionally tested panels.  
Figure 11: Section of bi-directionally tested panels  
Figure 12: In-plane panel before test  
Figure 13: Test set up of bi-directional test set up in MAST system
Comparison of In-Plane and Bi-Directional Performance

The damage state at the end of each test as well as the overall hysteretic behavior of the panels is shown in Figure 14 and Figure 15. Both sets of panels behaved in a similar fashion with flexural demands focusing on a single large crack at foundation level. Each of the panels yielded at 0.5% in-plane drift and experienced the onset of buckling of the longitudinal reinforcement at 1% drift, followed by longitudinal bar fracture and failure at 1.5% drift. There was little difference between the in-plane and bi-axial behavior of the panels, mostly because the strain demand on the single layer of reinforcement due to in-plane actions, coupled with the low axial load on the panel, meant that the main flexural crack was unable to fully close under out-of-plane demands, even with a 3:1 drift ratio. As such, there was very little additional demand put on the joint in the out-of-plane direction and the flexural crack did not propagate behind the insert head. It is possible that if a larger drift ratio of 4:1 was used, the performance of the joint may have been poorer, with vertical cracking behind the insert occurring. The only significant difference between in-plane and bi-axial loading was with the insert connection with 50 mm of cover behind the insert head. Due to slippage in the insert and the coupled out-of-plane demand, instead of simply cracking in the panel, the dowels in the foundation spalled the foundation cover concrete and the panel failure mechanism was due to large cracks in the panel at both insert layers in a brittle manner (Figure 14b). All other panels had only minor or no damage to the foundations.

Figure 14: Response of in-plane and bi-axial loading of threaded insert with shallow embedment (Panel TI12-C50)
(a) Damage state at end of in-plane loading  
(b) Damage state at end of bi-axial loading  
(c) Global in-plane hysteretic comparison for in-plane and bi-axial loading

Figure 15: Response of in-plane and bi-axial loading of threaded insert embedded to panel back (Panel TI12-C0)

ASSESSMENT METHODS

In order to investigate the relationship between the panel to joint strength ratio and breakout behind the threaded inserts, the joint strengths (M_{cb}) of the threaded insert panels in this and the Ma [10] study were calculated based upon the equations for anchorage pull out that are provided in both ACI 318-08 Appendix D [11] and NZSEE 3101:2006 [12]. This calculation represents current practice for the design of these connections. For panels in which inserts were spaced such that group action was in effect, the load was applied at the top row and the eccentricity between the anchor group centroid and load was accounted for. An alternative breakout strength was also calculated in which only the top row of inserts was assumed to be effective and assuming that the area of the failure cone was cut-off at the foundation level flexural crack (M_{cb}*). No strength reduction factors were applied to the calculated panel or joint strengths. The calculated joint strengths were compared to the nominal flexural capacity of the panel section and are summarized for all panels in Table 1.

The calculated joint capacities did not predict the breakout of any of the panels except for Panel Ma-1. The discrepancy in performance between the calculated joint strength and the observed test behavior suggests that the use of these anchorage equations is inappropriate for the design of such panel details. This inappropriateness was attributed to these equations being intended for the design of anchors or anchor groups in direct tension instead of predicting the interaction between the propagation of a flexural crack and the brittle failure of anchor pullout. Alternative design methods are required to more accurately estimate the strength and failure mode of the panel-to-foundation joints with dowel type connections.
### Table 1: Comparison of panel and connection strength of threaded insert panels

<table>
<thead>
<tr>
<th>Panel Name</th>
<th>Connection Description</th>
<th>Vert Reinf</th>
<th>$M_r$ Panel (kN-m)</th>
<th>$M_n$ Joint (kN-m)</th>
<th>$M_n/M_n$</th>
<th>$M_{cb}$ Joint* (kN-m)</th>
<th>$M_{cb}/M_n$</th>
<th>Breakout</th>
<th>Observed</th>
</tr>
</thead>
<tbody>
<tr>
<td>TI12-C50</td>
<td>TI12 HD12</td>
<td>16.1</td>
<td>20.0</td>
<td>1.2</td>
<td>25.2</td>
<td>1.4</td>
<td>yes</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TI12-C42</td>
<td>TI12 + Nail plate HD12</td>
<td>16.1</td>
<td>20.0</td>
<td>1.2</td>
<td>25.2</td>
<td>1.4</td>
<td>yes</td>
<td></td>
<td></td>
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<tr>
<td>TI12-C42-M</td>
<td>TI12 + Nail plate HD12</td>
<td>16.1</td>
<td>20.0</td>
<td>1.2</td>
<td>25.2</td>
<td>1.4</td>
<td>yes</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TI12-C50-FC</td>
<td>TI12 Full Cone HD12</td>
<td>16.1</td>
<td>123.2</td>
<td>7.6</td>
<td>120.1</td>
<td>7.4</td>
<td>no</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TI12-C50-FC-M</td>
<td>TI12 Full Cone HD12</td>
<td>16.1</td>
<td>123.2</td>
<td>7.6</td>
<td>120.1</td>
<td>7.4</td>
<td>no</td>
<td></td>
<td></td>
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<tr>
<td>TI16-C32</td>
<td>TI16 HD16</td>
<td>27.5</td>
<td>169.5</td>
<td>6.2</td>
<td>79.7</td>
<td>2.9</td>
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<td></td>
</tr>
<tr>
<td>TI16-C32-M</td>
<td>TI16 HD16</td>
<td>27.5</td>
<td>169.5</td>
<td>6.2</td>
<td>79.7</td>
<td>2.9</td>
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<td></td>
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<tr>
<td>TI16-C24</td>
<td>TI16 + Nail plate HD16</td>
<td>27.5</td>
<td>169.5</td>
<td>6.2</td>
<td>79.7</td>
<td>2.9</td>
<td>yes</td>
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<tr>
<td>TI16-C24-M</td>
<td>TI16 + Nail plate HD16</td>
<td>27.5</td>
<td>169.5</td>
<td>6.2</td>
<td>79.7</td>
<td>2.9</td>
<td>yes</td>
<td></td>
<td></td>
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<tr>
<td>TI16-C32-FC</td>
<td>TI16 Full Cone HD16</td>
<td>27.9</td>
<td>154.7</td>
<td>5.5</td>
<td>123.8</td>
<td>4.4</td>
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<tr>
<td>TI16-C32-FC-M</td>
<td>TI16 Full Cone HD16</td>
<td>27.9</td>
<td>154.7</td>
<td>5.5</td>
<td>123.8</td>
<td>4.4</td>
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<td></td>
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<tr>
<td>Ma-1</td>
<td>TI12 HD12</td>
<td>19.9</td>
<td>17.5</td>
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<td>17.9</td>
<td>0.9</td>
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<tr>
<td>Ma-4</td>
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<td>12.3</td>
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<td>17.9</td>
<td>1.5</td>
<td>no</td>
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</table>

*a TI = Threaded Insert; number following is diameter of starter bar
b M in panel name denotes monotonic loading
c All vertical reinforcing spaced at 270 mm
d $M_{cb}$* assuming breakout strength based on top row of inserts only with breakout cone cut-off at the foundation level flexural crack

### Conclusions

Testing of the out-of-plane, in-plane, and bi-axial response of dowel type foundation connections was performed on over thirty panels. The following behavior was identified along with the following recommendations:

- The single layer of reinforcement and low axial load on the panels resulted in a pinched hysteretic response in the out-of-plane direction. This response means that the use of a fixed-pinned cantilever model for design actions on these panels is inappropriate once yielding has occurred in the reinforcement.
- Conventional starter bars either in “L” or “U” shape performed adequately in the out-of-plane direction, being able to develop the nominal strength of the panel without joint failure.
- Threaded inserts with shallow embedment depths were all found to exhibit brittle failure of the joint initiated due to vertical cracking behind the insert head. It is recommended that the out-of-plane drift of these connections be limited to less than 2% to keep this failure from occurring.
- Three alternative connection details were tested and included increasing the embedment depth of the insert to the back of the panel, providing additional longitudinal reinforcement in the joint region, and confining the joint region. All alternative details were successful at maintaining joint integrity and nominal flexural capacity.
- The panels were found to sustain up to 1.5% drift when subjected to either in-plane or bi-axial loading, with damage focusing on a single flexural crack.
The use of anchor pull out equations in NZS 3101:2006 was found to be inappropriate for the design of threaded insert connections because the connection is not in direct tension but instead fails through the propagation of a flexural crack behind the insert.

Acknowledgements

Funding for this project was provided by the Building Systems Performance branch of the New Zealand Ministry of Business, Innovation, and Education, with project management from the UC Quake Centre. This project was partially supported by QuakeCoRE, a New Zealand Tertiary Education Commission-funded Centre. Wilco Precast and Precast NZ also provided significant support in providing drawings to develop prototype panels, panel construction, and testing space. Swinburne University of Technology helped to support testing costs and Reids ITW provided threaded inserts and financial support for panel and foundation construction. The authors would also like to extend their gratitude to Marjus Gjata, Sam Corney, James Burley, Tua Faitotoa, Sophie Burridge, Morgan Raby, Mark Casey, Hayden Wright, Peter Kendicky, Kevin Nievaart, Graeme Burnett, Michael Culton, and Scott Menegon for their assistance with experimental testing.

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