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RECENT TESTING AND DESIGN RECOMMENDATIONS FOR PRECAST CONCRETE PANEL-TO-FOUNDATION CONNECTIONS

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ABSTRACT

Precast concrete wall buildings represent a significant portion of the New Zealand building stock and range from low-rise industrial and commercial buildings to multi-storey residential and office buildings. Despite the prolific nature of this construction type, there is limited evidence of the seismic performance of existing connections between panels and other structural elements. In addition, previous earthquakes, including the Canterbury and Kiakoura 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 undertaken at the University of Auckland. The testing program focused on dowel type connections typically used for low-rise industrial or commercial buildings. The testing program consisted of over thirty singly reinforced concrete panels incorporating 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 will be described, 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 widely used construction form in low-rise industrial buildings. These buildings represent a substantial portion of the New Zealand building stock, and their post-earthquake functionality has important economic implications given their use as warehouse and distribution centres. 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 (Hamburger et al. 1988, Adham et al. 1996). While, earthquake reconnaissance following the 2010/2011 Canterbury earthquake sequence in New Zealand found that overall, precast and tilt-up concrete buildings performed adequately (Henry and Ingham 2011), vulnerabilities were identified in the out-of-plane response of dowel type panel-to-foundation connections for panels with a single layer of vertical reinforcing, particularly for panels that utilized threaded inserts embedded in the panels to connect starter bars to the foundation (SESOC 2013). 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 panel construction because the starter bars can be screwed into the panels after they were erected and thus avoided the need to bend bars for transport and storage thus reducing labour and time on site prior to pouring of the foundation (Beattie 2007). 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. The limited testing on this type of connection performed by Ma (2000) showed that while no vertical cracking in the panel joint region was observed, the connection could not sustain the nominal moment capacity of the panel in the joint opening direction. To investigate the strength and deformation capacity of typical threaded insert connections, a series precast panel to foundation connections of different nominal panel and joint connection strengths were subjected to out-of-plane loading, in-plane, and bi-directional loading. Performance of threaded insert connections was compared to connections without inserts that utilized starter bars with conventional hooked bar anchorage, as these are also a common connection type with potentially similar vulnerabilities as the threaded insert connections. Cases in which connection details were found to provide inadequate behaviour, alternative details were proposed and tested. Finally, the results from the testing program were used to evaluate the ability of existing design equations in NZS 3101:2006 to predict the failure strength of this insert type.

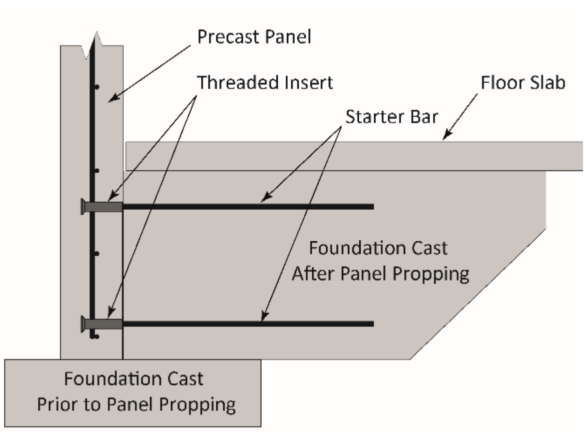


Figure 1: Schematic representation of a dowel type precast panel to foundation connection utilizing threaded inserts.

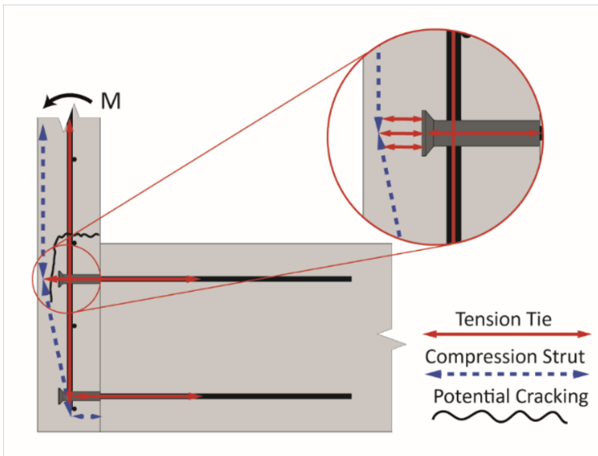


Figure 2: Strut and tie representation of load path for panel to foundation connection using threaded inserts with a shallow embedment depth

OUT-OF-PLANE PERFORMANCE

Twenty seven panels utilizing different connection details were tested in the out-of-plane direction to determine the performance of the panel to foundation connection. All panels utilized a single layer of either HD12 or HD16 reinforcing at 270 mm vertical spacing to correspond with the minimal reinforcing ratios typically used for these panels. The test set up and reinforcement details for the out-of-plane tests are detailed in Burley et al. (2014) and Burridge et al. (2015). An overview of the performance of these details subjected to out-of-plane loading is provided in the following sections, with a focus on the ability of the connection to limit damage in the joint region and develop the nominal moment capacity of the panel.

Control Specimen with Starter Bars Bolted Through the Panel

A panel with a panel-to-foundation connection in which the starter bars were bolted through the panel to an RHS section, was used as a control specimen to investigate the behaviour of the panel without the influence of the joint damage. This panel, Panel BLT-C0, which is shown in Figure 3, was subjected to cyclic loading up to 6.75% drift. As expected the joint region was undamaged (Figure 3) and a limited number of flexural cracks developed in the bottom 1.0 m of the panel above the foundation level. The hysteretic moment-rotation behaviour of the panel is shown in Figure 4. The hysteretic response of the panel was stable with the panel being able to develop its nominal moment capacity in both joint-opening and joint-closing directions. Significant pinching was observed in the hysteretic response as can be seen in the highlighted 4.5% drift cycle shown in Figure 4. This pinching is a result of inelastic elongation of the single layer of reinforcement and the low axial load on the panel. As the panel is subjected to inelastic demand on the reinforcement, the panel does not have enough axial load to close the flexural cracking that has occurred and as such must undergo a rotation of greater magnitude than the previous cycle before crack closure occurs and the panel can resist flexural demands. This behaviour makes the panel foundation effectively pinned once yielding of the reinforcement has occurred.

Figure 3: Damage of Panel BLT12-C0 at 4.5% drift in the joint-opening direction

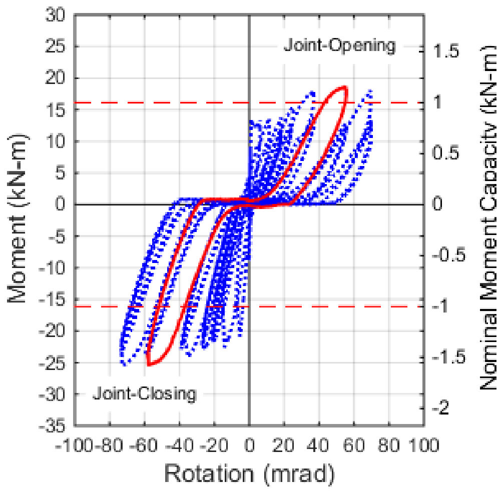


Figure 4: Global Moment-Rotation behavior of Panel BLT12-C0 with 4.5% drift cycle highlighted.

Conventional Starter Bars

Two panels were tested that utilized conventional starter bars either from D12 bars bent into a “U” or “hairpin” shape, as is shown in Figure 5, or in two “L” shaped hooks with 600 mm long returns into the panel. Overall, the conventional reinforced starter bar connections performed well, with minimal or no damage occurring in the joint region and the panel developing its nominal moment capacity in both the joint-opening and joint-closing directions. The “L” shaped hooks were able to protect the joint by the hook returns effectively tripling the vertical reinforcement in the 600 mm above the foundation level. Flexural cracking was forced above the location of the hook return and as such, the joint was protected from excessive rotational demands or damage.

The “U” bar connection, also known as a “hairpin” connection also performed better than expected. Because the connection had similar embedment depth as the potentially vulnerable threaded insert connections such as those shown in Figure 2, it was expected that vertical cracking in the joint would cause the panel to lose flexural capacity. Instead the panel exhibited only minor vertical cracking in the joint (Figure 6) and had a stable hysteretic response and reaching nominal moment capacity in both the joint-opening and joint-closing directions as can be seen in Figure 7.

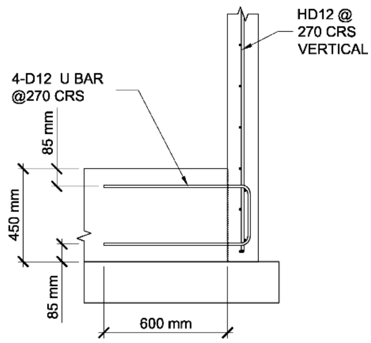


Figure 5: Detail of a conventional starter bar connection (Panel U12-C50)

Figure 6: Joint condition of a conventional starter bar connection (Panel U12-C50)

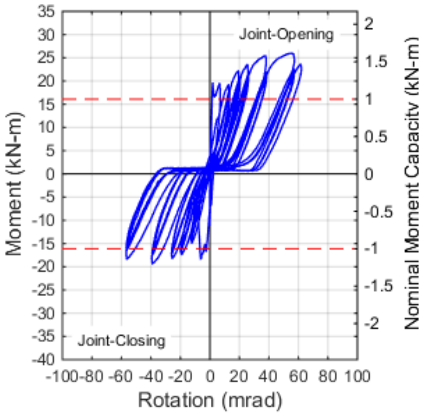


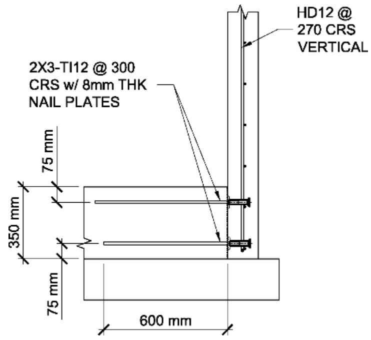
Figure 7: Global moment-rotation response of a conventional starter bar connection (Panel U12-C50)

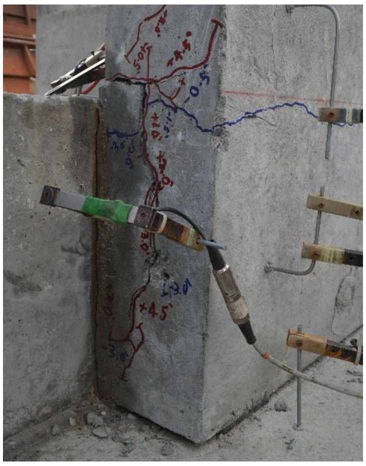
Threaded Inserts with Shallow Embedment

Twelve panels with shallowly embedded threaded inserts were tested with either monotonic or cyclic out-of-plane loading. The panels consisted of either a single layer of HD12 reinforcing with two layers of 12 mm diameter threaded inserts as is shown in Figure 8, or a single layer of HD16 reinforcing with two layers of 16 mm diameter threaded inserts. Insert spacing and foundation depth was varied to be consistent with current design and construction practices using this connection detail. Finally, the use of plastic nail plates to hold the inserts during casting was also investigated.

All panels performed similarly with the all but two panels exhibiting significant vertical cracking in the joint region (Figure 9) followed by degradation in strength and stiffness after 20 mrad of rotation in the joint-opening direction (Figure 10). It was found that typically the HD12 panels were able to develop the nominal moment capacity in both directions before cracking in the joint caused the panel to lose strength. None of the HD16 panels were able to reach the nominal moment capacity in the joint-opening direction during cyclic loading.

It was also observed that the use of the nail plate during casting of the panel had a negative impact on the out-of-plane performance of the panel when compared to the panels that did not utilize nail plates. Without nail plates, the insert opening was at the interface between the panel and the foundation. During joint-opening actions, slip in the insert allowed this interface to open up and as such there was less tensile demand on the starter bar for a given drift level. The panels that utilized nail plates were unable to open up the panel-foundation interface to the same degree because the void left by the nail plate was filled with concrete during the foundation pour effectively locking in the starter bars and creating greater tension demand in the concrete in the joint region.





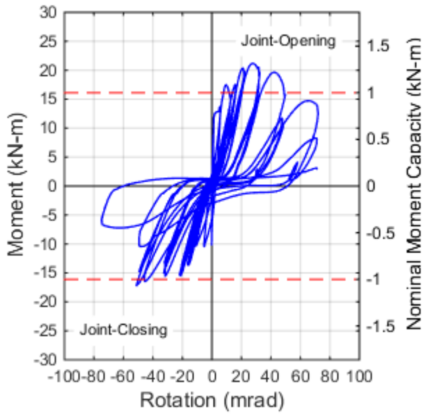


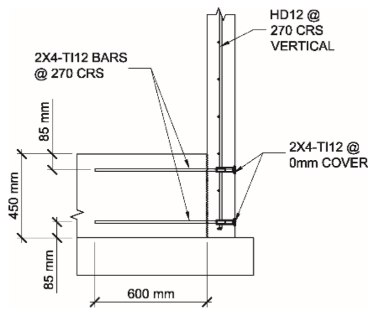
Figure 8: Detail of shallowly embedded threaded insert connection (Panel TI12-C42)

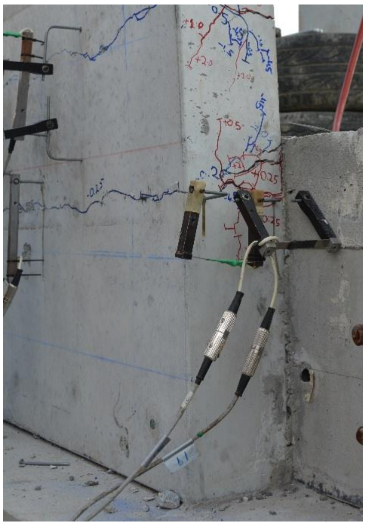
Figure 9: Joint damage to shallowly embedded threaded insert connection (Panel TI12-C42)

Figure 10: Global moment-rotation response of shallowly embedded threaded insert connection (Panel TI12-C42)

Alternative Detailing

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 reinforcing in the joint region to force damage further up the panel height, and providing confinement in the joint region.





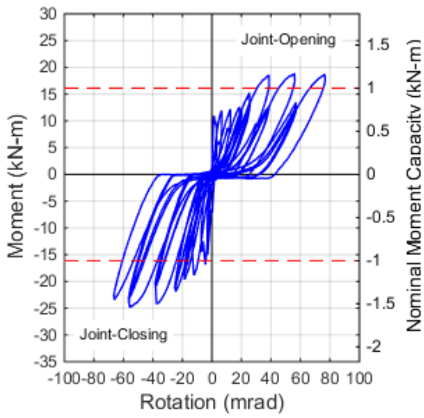


Figure 11: Detail of threaded insert embedded to panel back (Panel TI12-C0)

Figure 12: Joint condtion of threaded insert embedded to panel back (Panel TI12-C0)

Figure 13: Global moment-rotation response of threaded insert embedded to panel back (Panel TI12-C0)

The first series of alternative details was to increase the embedment depth of the threaded insert to allow for the load path shown in Figure 2 to have a tension tie into the foundation that did not involve concrete acting in tension. This connection was developed to mimic the good performance that was shown in the bolted through control specimen, and included two variations: one in which the threaded insert was embedded to the back of the panel with 0 mm of cover behind the insert head (Figure 11), and one in which there was 15 mm of cover behind the insert head. Both connections performed in a similar manner in which no damage was observed in the joint region (Figure 12) and the connection was able to develop the nominal flexural capacity of the panel in both the joint-opening and joint-closing direction with a stable hysteretic response as shown in Figure 13.

The second series of alternative details was the use of additional reinforcement in the joint region to force damage into the panel and protect the joint. The use of both proprietary headed studs and conventional reinforcement bent into a “link bar” were investigated with the details of the link bar provided in Figure 14. This connection variation was successful in protecting the joint from damage (Figure 15) and allowing the panel to develop the nominal moment capacity in both joint-opening and joint-closing directions (Figure 16). It should be noted that the additional reinforcement added to the joint region effectively doubled the longitudinal reinforcement ratio in that region when compared to the rest of the panel.

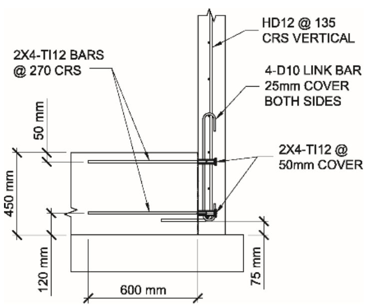


Figure 14: Detail threaded insert connection with strengthend joint region (Panel TI12-C50-LB-V135)

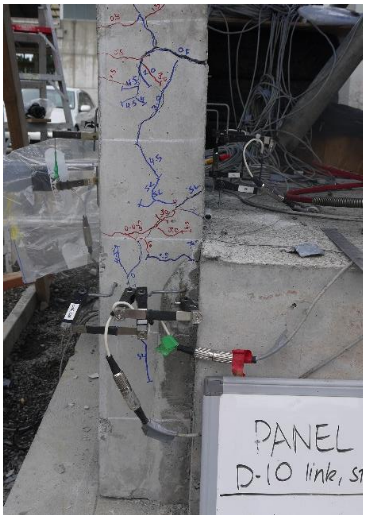


Figure 15: Joint condition of a threaded insert connection with strengthend joint region (Panel TI12-C50-LB-V135)

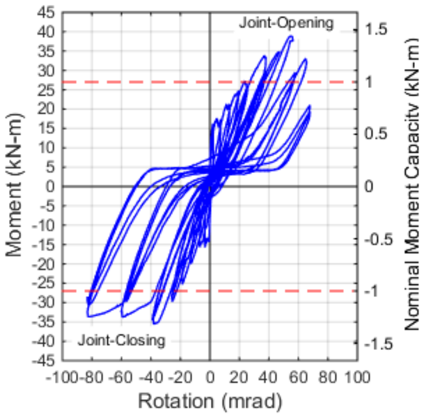


Figure 16: Global moment-rotation response of a threaded insert connection with strengthend joint region (Panel TI12-C50-LB-V135)

The final alternative detail variation was the use of confining stirrups around the threaded insert, which was cast with 50 mm cover behind the insert head as is shown in Figure 17. This connection was successful at keeping damage located to outside the joint region, with flexural cracking occurring just above the joint stirrup (Figure 18). Despite the well performing joint condition after the test, the hysteretic response of the panel showed that it was unable to develop the nominal flexural capacity in the joint-opening direction while exhibiting significantly higher than expected joint-closing flexural capacity (Figure 19). Post-test investigation and analysis revealed that the single layer of reinforcing was most likely offset by 10 mm due to the wrong reinforcement chair being used. This misplacement of reinforcement highlights the sensitivity of singly reinforced panels to construction tolerances.

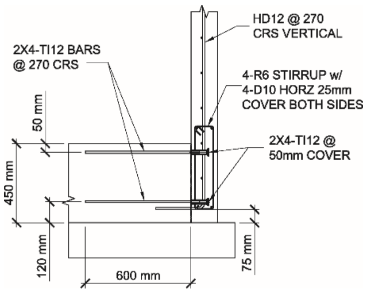


Figure 17: Detail threaded insert connection with confined joint region (Panel TI12-C15-STRP)

Figure 18: Joint condition of a threaded insert connection with confined joint region (Panel TI12-C15-STRP)

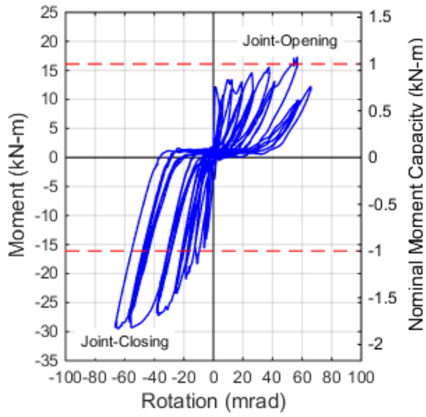
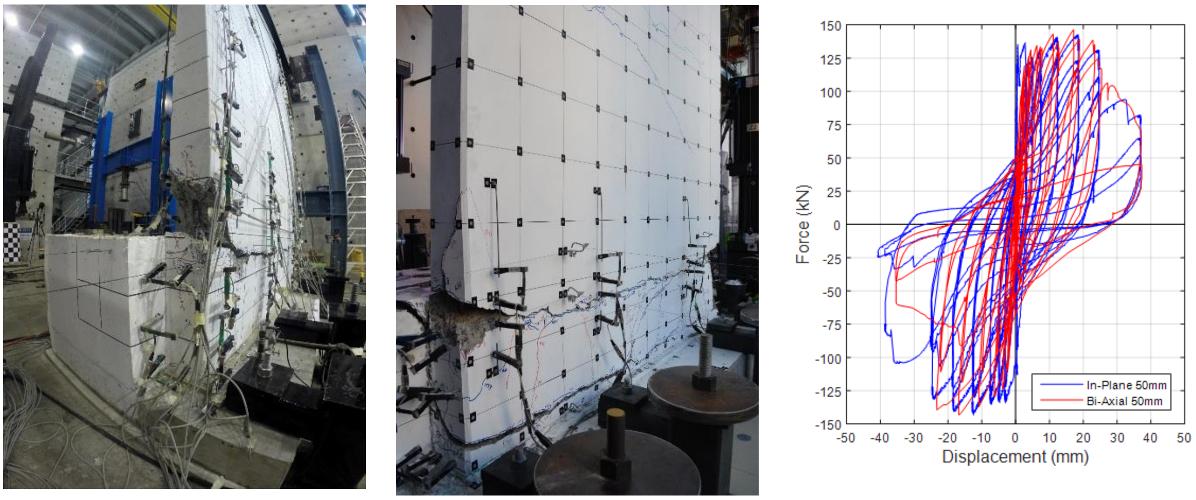


Figure 19: Global moment-rotation response of a threaded insert connection with confined joint region (Panel TI12-C15-STRP)

IN-PLANE & BI-AXIAL PERFORMANCE

With the out-of-plane behaviour of several common and alternative panel-to-foundation connections established, the behaviour 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. In-plane and bi-axial testing was performed on two types of threaded insert connections, one with shallow embedment constructed as per current practice with 50 mm cover behind the insert and one alternative detail in which the insert head was pushed to the back of the panel. An axial load ratio of 0.04% was applied to the wall to simulate the self-weight of a 10 m tall panel. The panels were subjected to increasing in-plane cyclic drift and for the bi-axial panels, a drift ratio of 3:1 was used for the out-of-plane to in-plane drift demands, with the out-of-plane action occurring first in the loading protocol.

The damage state at the end of each tests as well as the overall hysteretic behaviour of the panels is shown in Figure 20 and Figure 21. 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 behaviour 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 main flexural crack was unable to fully close under out-of-plane demands, even with a 3:1 drift ratio. 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 20b).

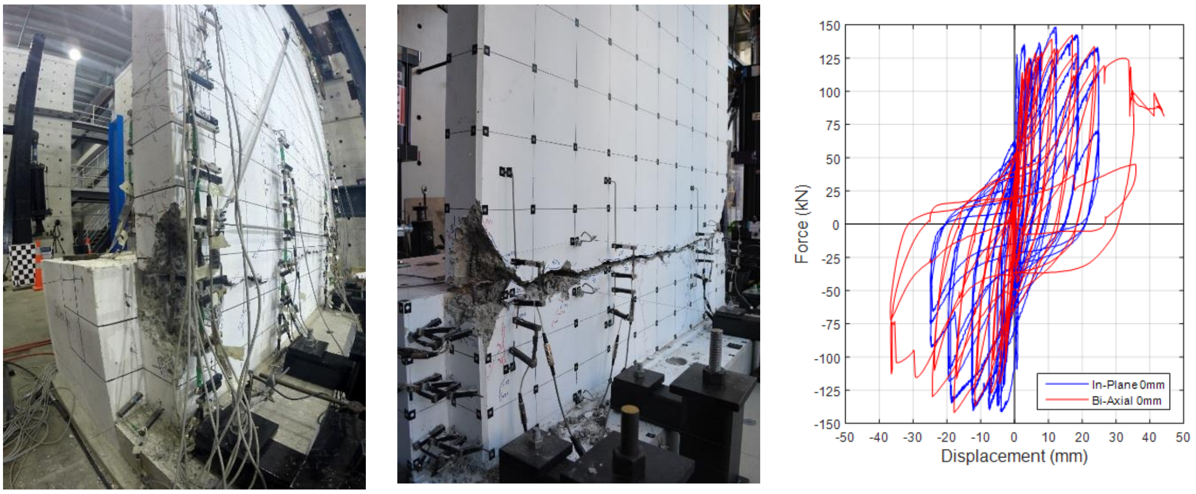


(a) Damage state at end of in-plane loading

(b) Damage state at end of bi-axial loading

(c) Global in-plane hysteretic comparison for in-plane and bi-axial loading

Figure 20: 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 21: 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 (Mcb) of the threaded insert panels in this and the Ma (2000) study were calculated based upon the equations for anchorage pull out that are provided in both ACI 318-08 Appendix D (ACI 2008) and NZSEE 3101:2006 (Standards New Zealand 2006). 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 (Mcb*). 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 behaviour 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 the 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

Panel Name	Connection Description	Vert Reinf	Mn Panel (kN-m)	Mcb Joint (kN-m)	Mcb/ Mn	Mcb* Joint[d] (kN-m)	Mcb*/Mn	Breakout Observed
TI12-C50	TI12	HD12	16.1	20.0	1.2	25.2	1.4	yes
TI12-C42	TI12 + Nail plate	HD12	16.1	20.0	1.2	25.2	1.4	yes
TI12-C42-M	TI12 + Nail plate	HD12	16.1	20.0	1.2	25.2	1.4	yes
TI12-C50-FC	TI12 Full Cone	HD12	16.1	123.2	7.6	120.1	7.4	no
TI12-C50-FC-M	TI12 Full Cone	HD12	16.1	123.2	7.6	120.1	7.4	no
TI16-C32	TI16	HD16	27.5	169.5	6.2	79.7	2.9	yes
TI16-C32-M	TI16	HD16	27.5	169.5	6.2	79.7	2.9	yes
TI16-C24	TI16 + Nail plate	HD16	27.5	169.5	6.2	79.7	2.9	yes
TI16-C24-M	TI16 + Nail plate	HD16	27.5	169.5	6.2	79.7	2.9	yes
TI16-C32-FC	TI16 Full Cone	HD16	27.9	154.7	5.5	123.8	4.4	yes
TI16-C32-FC-M	TI16 Full Cone	HD16	27.9	154.7	5.5	123.8	4.4	yes
Ma-1	TI12	HD12	19.9	17.5	0.9	17.9	0.9	yes
Ma-4	TI12	HD12	12.3	17.5	1.4	17.9	1.5	no

[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] Mcb* 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 behaviour 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 reinforcing 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 as the connection is not in direct tension but instead fails through the propagation of a flexural crack behind the insert.

ACKNOWLEDGMENTS

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