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IMPROVED DETAILING OF PRECAST CONCRETE PANEL TO FOUNDATION CONNECTIONS TO WITHSTAND OUT-OF-PLANE EARTHQUAKE LOADS

S.M. Burridge, M.P. Casey, M.L. Raby, H.D. Wright, L. Hogan, R.S. Henry, J.M. Ingham

Department of Civil and Environmental Engineering, University of Auckland

ABSTRACT

Concerns have previously been raised regarding the seismic performance of slender precast panel to foundation connections, in particular the apparent lack of a viable load path when the connection is subjected to an out-of-plane opening moment. Previous testing in 2014 showed that current New Zealand standard practice regarding the use of threaded inserts in precast panels led to less than optimal performance when subjected to out-of-plane loading. To address the concerns raised during these tests, a second series of tests has been planned to investigate the out-of-plane response of modified panel to foundation connection details, including threaded insert details and details using conventional cast-in reinforcement starter bars. Details of the 16 planned experimental tests are described followed by preliminary results from the two panels tested to date.

INTRODUCTION

Precast concrete panels are frequently used in the New Zealand for a wide variety of projects within the commercial, industrial and residential sectors. The use of precast panels are often preferred over other construction methods as they offer significant reductions in construction time, and are readily available from specialist precast companies. A critical aspect of precast concrete construction is the detailing of the connections. Precast concrete panel connection details vary considerably depending on the size, shape, and intended purpose of the project in which they are being used. For industrial buildings, it is common to use slender singly reinforced panels with wall-to-foundation connections that rely on either bent starter bars or threaded inserts (Seifi et al. 2015). However, concerns were raised by the Structural Engineering Society of New Zealand (SESOC) following the 2010/2011 Canterbury Earthquakes regarding the seismic performance of shallow embedded anchors in precast concrete panels (SESOC 2013).

A study was initiated in 2014 in collaboration with industry to investigate the out-of-plane performance of panel to foundation connections that used threaded inserts (Burley et al. 2014). The testing conducted by Burley et al. showed that current New Zealand standard practice regarding the use of threaded inserts in precast panels led to less than optimal performance, with the pattern of damage observed during the tests consistent with the anticipated deficiencies in the panel-to-foundation connection. However, despite the response being sub-standard, the observed force-displacement characteristics were not classically brittle in nature.

To address the concerns raised by the Burley et al. (2014), a follow up series of test was initiated to investigate the out-of-plane response of modified panel to foundation connection details. The objective of this research was to conduct a comprehensive experimental investigation for a variety of panel-to-foundation connections subjected to out-of-plane earthquake actions and included both threaded insert details and details using conventional cast-in reinforcement starter bars.

BACKGROUND

The 2014 study reported by Burley et al. (2014) utilised information about panel to foundation connection design in constructed buildings to test common industry designs for industrial precast concrete panels that used threaded inserts. A total of 12 panels were tested with specimen variations including loading regime (cyclic and monotonic), foundation depth, the use of nail plates, and insert and reinforcement size. All test panels included a double layer of insert connections with either RB12 or RB16 inserts and a foundation depth of either 350 mm or 710 mm. The test panels were created to simulate the lower portion of a full scale 10 m high concrete wall. By applying a point load at the top of a 2.5 m high test panel, the lateral load distribution on a full scale 10 m high panel could be approximated, as shown in Figure 1.

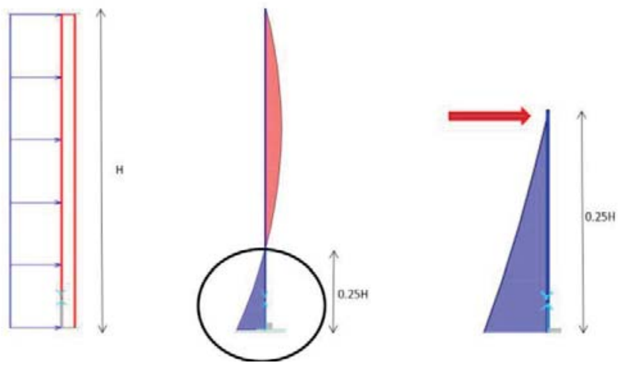


Figure 1 – Lateral load and bending moment distribution for test walls

During almost all of the Burley et al. tests the flexural crack at the base of the panel propagated vertically down into the joint region, as shown in Figure 2. The vertical crack appeared to pass behind the threaded inserts, which were embedded slightly over halfway onto the panel thickness. The cracking in the joint severely degraded the response when subjected to higher drift levels, and was considered undesirable when compared to the ductile flexural response of the panel that was intended from the design.

Figure 2 - Panel vertical crack propagation

A typical moment-rotation response recorded for the test panels is shown in Figure 3. Many panels did not achieve their full nominal flexural capacity prior to the connection degrading.

Additionally, during larger drift cycles the hysteresis response displayed severe ‘pinching’ with the connection damage resulting in a pinned behaviour when subjected to cyclic loading.

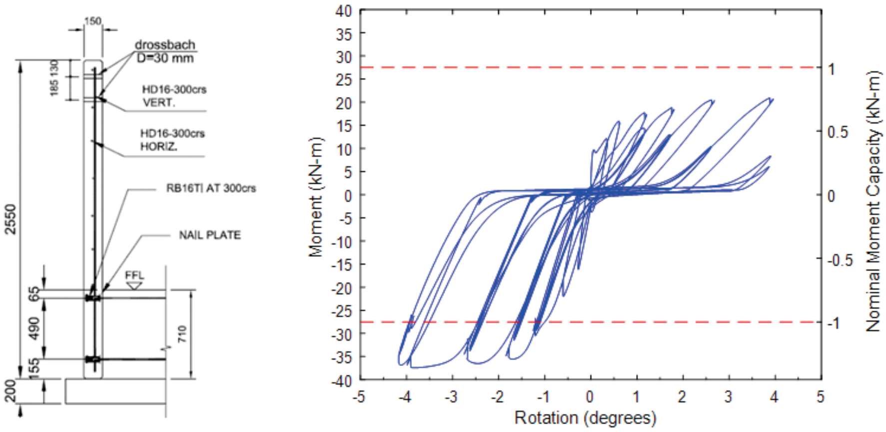


Figure 3 - Moment rotation response for test panel 6B

The key findings from the Burley et al. tests included:

- Failure of connection occurred before the full flexural strength of the panel could be achieved.
- The moment rotation hysteresis response of the panels subjected to cyclic loading displayed severe pinching, with the connection responding like a pin during large drifts.
- The connections with a deeper foundation depth performed better than those with the shallow foundation as it allowed greater cone development and less cone interference.
- The use of nail plates to increase the insert embedment had no significant influence on the panel behaviour.

DEMOLITION OF PREVIOUS TEST PANELS

At the start of this study, damage to the Burley et al. (2014) test panels was inspected to better understand the failure mechanisms within the connection. To do this the 12 test panels were carefully deconstructed, while photographic evidence of the failure plane within in the panel and connection was collected. It was confirmed that the vertical crack in the connection region propagated behind the threaded inserts, resulting in a cone type failure developing in most of the test panels, as shown in Figure 4.

Figure 4 - Cone breakout failure observed during panel demolition

IMPROVED CONNECTION DESIGNS

The results from the Burley et al. (2014) tests indicated that the standard double layer of threaded insert panel-to-foundation connections performed poorly during cyclic loading. The test results highlighted the need for further research into the cyclic performance of all current connection details, including both threaded inserts and bent starter bars. Additionally, improved connection details to overcome the weaknesses in existing threaded insert connections needed to be designed and verified. To investigate these variations, a total of 16 panel-to-foundation connection designs were developed for testing.

To determine a solution to the problems associated with shallow embedded connections it is crucial to understand the load path, in order to identify potential weaknesses. The load path associated with shallow embedded connection subjected to an opening moment is shown in Figure 5. Due to the shallow embedment depth of the threaded inserts, there is a region behind the head of the insert that relies on tensile strength of the concrete to complete the tension tie. The region of concrete in tension results in the flexural crack at the base of the panel propagating vertically behind the insert, as shown in Figure 5 and during the Burley et al. (2014) tests.

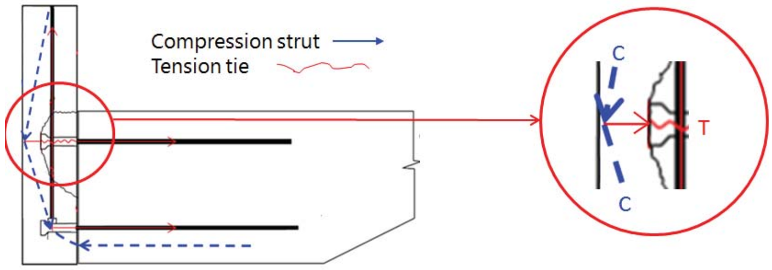
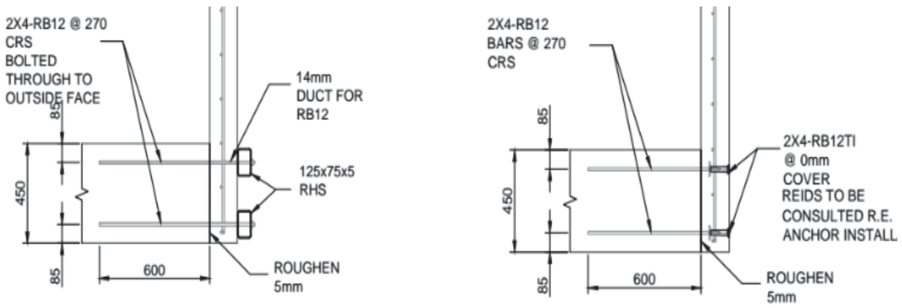


Figure 5 - Strut and tie model for threaded insert connection

Threaded insert embedment depth

From the load path analysis in Figure 5, the shallow embedment depth of the inserts clearly contributed to the failure of the connection. Therefore, all standard test panels with threaded inserts were designed with the anchors embedded to a distance of 15 mm from the back face of the panel. The exception to this embedment depth was Panels 1 and 2, which use a bolted through Reid bar, and threaded inserts with zero cover respectively, as shown in Figure 6. These two connections were included as an idealised base case, although it is recognised that in practice neither of these connections would be particularly viable due to cover requirements and architectural specifications.

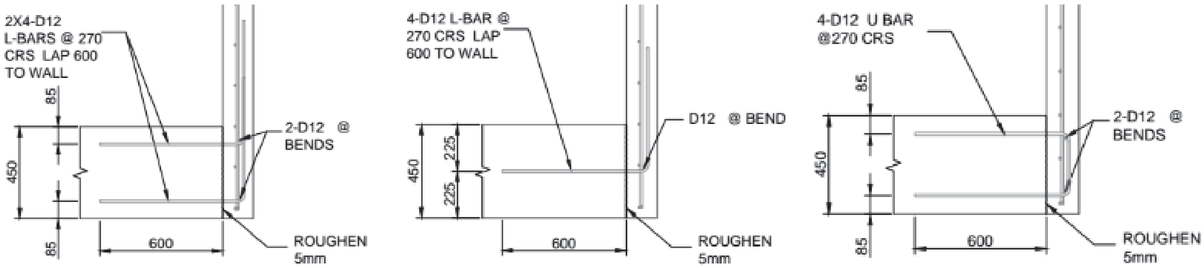


(a) Panel 1 (b) Panel 2

Figure 6 – Test panels: Embedment depth

Starter bar reinforcement

Cast in starter bar connections involve reinforcement extending from the panel into the foundation and anchored in the panel connection with either a 90° bend or by using a U-shaped bar. The use of starter bar connection in slender panel connections is common (Seifi et al. 2015), but there are potential issues due to the bars being bent parallel to the panel for transportation and re-bent on-site (Ma, 2000). When applied in a connection between a wall and a slab, 90^o hook bars also have a disadvantage in terms of the bar size available, due to minimum bend radius requirements (De Vries, 1996). Despite the issues surrounding these types of connections, directly comparing the performance of cast in starters against threaded inserts will be highly beneficial in determining which method provides a superior connection. There panels with starter bars have been designed for testing, including both 90^o bend and U-shaped bars, as shown in Figure 7.

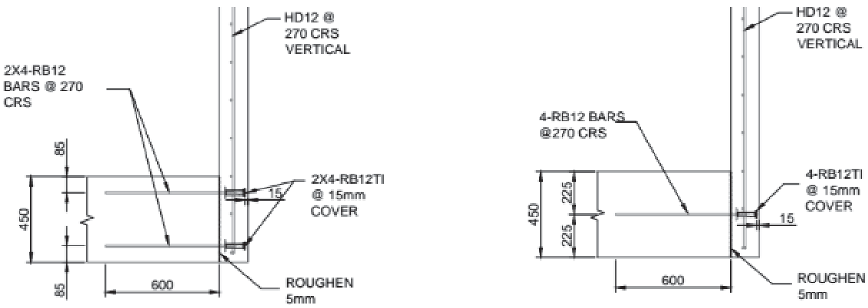


(a) Panel 3 (b) Panel 4 (c) Panel 5

Figure 7 - Test panels: Starter bars

Single and double row of inserts

After conducting their research, Burley et al. (2014) hypothesised that the relatively shallow depth of the foundation, and hence the closeness of the inserts to each other, resulted in an overlapping of the failure cones. This would have resulted in increased stresses in the concrete at the overlapping point, perhaps leading to premature pull out failure of the insert. To determine the effect of the overlapping failure cones, two test panels were designed with both a single and double row of inserts, as shown in Figure 8. The connections with a single row of inserts should not exhibit any overlapping cone behaviour, but no other factors will be changed, allowing for a direct comparison with a double row insert connection.

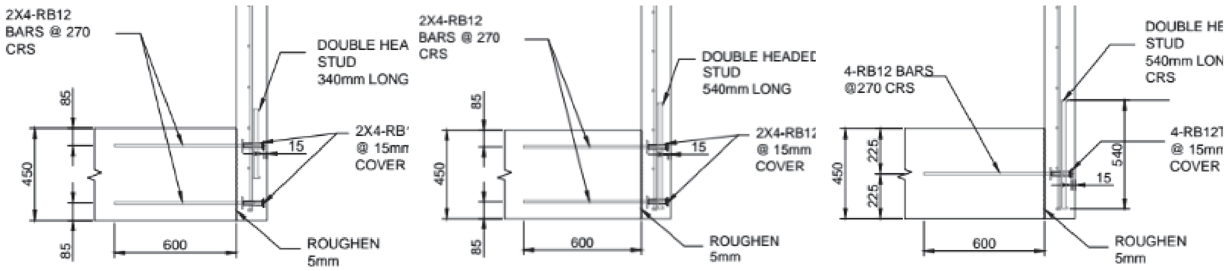


(a) Panel 6 (b) Panel 11

Figure 8 - Test panels: Single vs. double row of inserts

Additional double headed studs

Selected connection details will incorporate double headed studs of either 340 mm or 540 mm length in the wall panel, as shown in Figure 9. The headed studs are designed to strengthen the wall panel at the bottom close to the connection to ensure that the connection response is controlled by ductile flexural response of the panel rather than the previously observed connection failure. These double headed studs will be used with various combinations of wall reinforcement, number of rows of the connection inserts, and various insert alignment relative to the panel reinforcement.

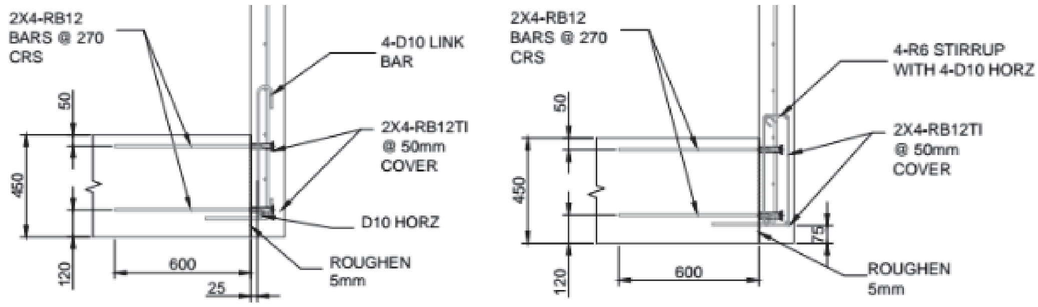


(a) Panel 12 (b) Panel 13 (c) Panel 10

Figure 9 - Test panels: Double headed studs

Additional joint reinforcement

A further three panels were designed with additional reinforcement in the connection region, including two that incorporated stirrups and one with a hooked link bar, as shown in Figure 10. The aim of the stirrups was to confine the concrete at the connection, and to provide reinforcement to control any potential cracks in the joint region. Both the stirrup and hooked link bar details also provide greater panel reinforcement and lever-arm at the connection (in the case of the hooked link bar, only in one direction), theoretically making the panel stronger in this region.



(a) Panel 14 (b) Panel 15/16

Figure 10 - Test panels: Additional reinforcement

PERLIMINARY TEST REULTS

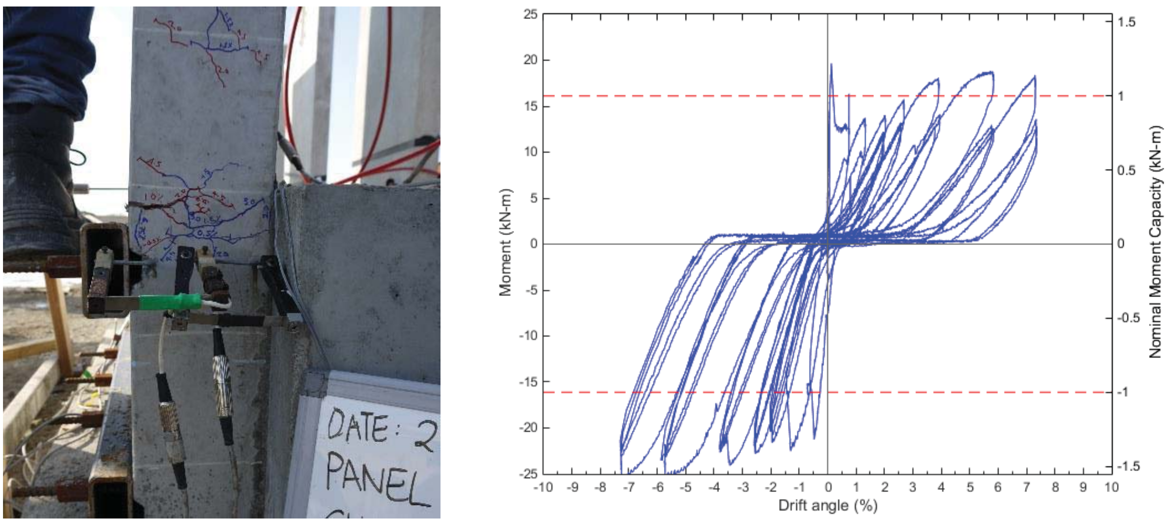
The test setup is shown in Figure 11, and was based on the test setup used by Burley et al. (2014) during the previous out-of-plane panel tests. Displacement controlled, quasi-static, reverse cyclic loading was applied at a height of approximately 2.5 m above foundation using a 500 kN hydraulic jack. Testing commenced with a single cycle at each level until the panel reached its calculated nominal yield strength, after which three cycles were conducted for each drift level.

Lateral displacements were measured close to the top of the panel and both above and below the panel-to-foundation interface using a draw wire and two LVDT's. Crack widths, joint rotation and separation of the panel to foundation connection were monitored by using portal gauges near the panel base. Additionally, strain gauges were placed on the reinforcement that extended into the foundation.
Currently, Panels 1 and 12 have been tested, with the remaining to be tested shortly. Panel 1 that used the bolted through connection was tested first as it was considered most likely to give the desired flexural mechanism at the base of the panel.

Figure 11 - Test setup

Panel 1

Panel 1 was designed as the ‘ideal’ connection detail and performed as expected due to the foundation connection reinforcement being well developed from the outside face of the panel. Initial flexural cracks formed at the base of the panel during cycles to 0.5% drift, as shown in Figure 12a. As the drift levels increased, additional flexural cracks formed higher up the panel, at approximately 200 mm centres. The highest crack occurred at approximately 600 mm above the top of the foundation. At 2% drift, the two largest flexural cracks were approximately 1.5 mm to 2.0 mm wide. By 4.5% drift, 4 mm cracks were observed immediately above the top rectangular hollow section, and concrete crushing was observed on the reversing cycle. The response was dominated by flexural deformations in the panel itself, with no damage occurring within the connection joint region.



(a) Panel cracking (b) Moment-drift hysteresis response

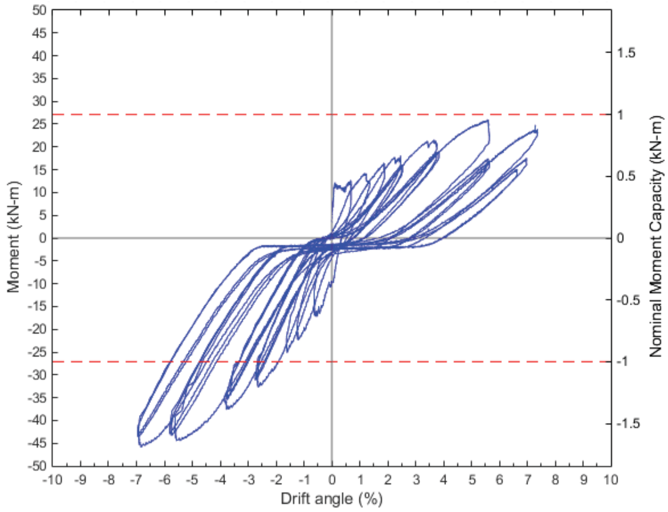
Figure 12 - Panel 1 test results connection cracking and moment-drift hysteresis plot

Figure 12b shows the measured moment-drift response for Panel 1. The top right quadrant is the push direction (away from the foundation), while the bottom left quadrant is the pull direction (towards the foundation). Panel 1 reached its nominal moment capacity of 16.1 kNm in both positive and negative loading directions. It was slightly stronger in the negative loading direction, reaching 25.4 kNm, while only reaching 19.6 kNm in the positive direction. When the panel was loaded repeatedly to the same drift displacement, the first cycle showed higher moment resistance than the subsequent cycles, however the strength was reasonably stable in the second and third cycles. The hysteresis response exhibited some energy dissipation due to the flexural behaviour of the panel, but noticeable pinching was observed, as is characteristic of single reinforced panels with no axial load.

Panel 12

Panel 12 was one of the threaded inserts connections with additional double headed studs. As with Panel 1, flexural cracks initiated at the panel base during early drift cycles. During larger drift cycles additional flexural cracks formed up the height of the panel and crack widths widened. The highest flexural crack occurred at approximately 1200mm above the foundation. No significant damage was observed to the panel connection joint region, apart from a slight vertical hairline crack propagating from the horizontal crack at the level of the top insert during cycles to 4.5% drift, as shown in Figure 13a.

(a) Panel cracking



(b) Moment-drift hysteresis response

Figure 13 - Panel 12 connection cracking and moment-drift hysteresis plot

As can be seen by the moment-rotation plot in Figure 13b, the panel nominal moment capacity of 27.1kNm was easily achieved in the negative pull direction (45.9kNm), but was not reached in the positive direction. It should be noted that Panel 12 was designed with a higher nominal moment capacity than panel 1 due to a larger amount of panel reinforcement. The cyclic hysteresis response was less stable that that observed for Panel 1, and a slight reduction in strength was observed at the last drift cycle in the positive push direction.

A connection failure was not exhibited in panel 12. In comparison with the test performed by Burley et al. (2014) results, it is likely that the deeper embedment depth of the inserts into the panel and the addition of the headed students improved the performance of the connection region. Further tests on the remaining panels containing this detail will help determine the effect of panel strengthening at the connection.

CONCLUSIONS

A series of 16 panel-to-foundation connections have been designed to investigate improved connection details for slender precast concrete panels. Initial test results have indicated that the desired flexural response in the panel can be achieved prior to connection failure if appropriate detailing is used. The effect of insert embedment depth appears to be a key factor in ensuring the desired behaviour is achieved as per the design. The 14 remaining tests are still to be undertaken and will provide a greater understanding of the performance of threaded insert and starter bar wall-to-foundation connections.

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