Experimental Testing of Precast Connections for Jointed Precast Concrete Building Cores

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Abstract

The majority of low, mid and high-rise buildings in Australia utilise reinforced concrete (RC) walls as their primary lateral load resisting system. In the last 10 years there has been a dramatic uptake in precast construction, which has resulted in most low and mid-rise buildings less than about eight storeys adopting jointed precast concrete building cores as opposed to traditional cast in-situ construction. These cores are typically connected together using welded stitch plate connections. Despite the widespread use of these connections and this type of construction, very little experimental and analytical research has been performed. This paper outlines a recent experimental program where an industry standard welded stitch plate connection was tested to assess its strength and stiffness. In addition to the welded stitch plate connection, two innovative new connections were tested. The innovative new connections have been designed such that the need for welding is eliminated on site. An overview of the experimental program and results are presented within.

Keywords: RC walls, wall testing, building cores, precast building core connections.

1 Introduction

Reinforced Concrete (RC) walls have widely be used in Australia as the primary lateral load resisting system for low, mid and high-rise buildings for many years (Menegon et al. 2017b). Traditionally RC walls were constructed as cast in-situ elements, however in more recent times there has been a widespread shift towards precast concrete walls, which are cast off-site in a precast yard or factory and later transported and erected on site. Precast concrete walls have primarily been adopted to replace cast in-situ walls in low and mid-rise buildings.

Rectangular precast concrete walls are commonly assembled around lift shafts and stairwells to form jointed precast concrete building cores. The panels are connected vertically to other panels above or below using grout tube connections (refer Menegon et al. (2017b)) and connected horizontally to adjacent panels using welded stitch plate (WSP) connections. The WSP connection typically consists of a structural steel equal angle (‘inside fixed’) or flat bar (‘outside fixed’) section that is site welded to cast-in plates on each respective panel. The cast-in plates have shear studs welded to the rear side of the plate interlocking with the concrete (i.e. the RC panel).

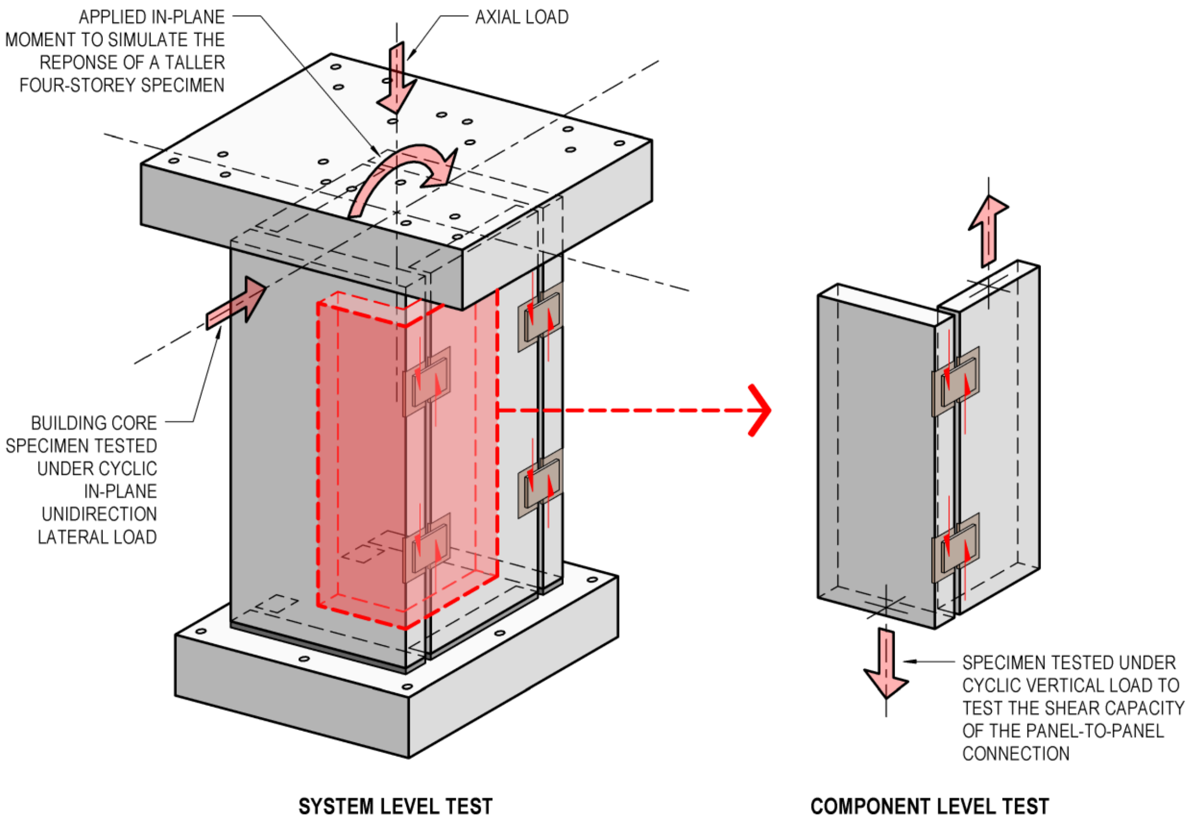
The primary purpose of the horizontal panel-to-panel WSP connection is to transfer vertical shear force between adjacent panels to allow composite action to be developed, so the individual panels can act together as one section (i.e. a building core) under lateral load. The strength of the WSP connection required is dependent on the configuration of the core and the intensity of the lateral load design actions the building is required to be designed to withstand. When the vertical shear forces are greater than the maximum capacity of the WSP connection, wet joints are usually adopted. Wet joints consist of a cast in-situ portion of concrete that is poured between two adjacent precast panels (as shown in Figure 1). Wet joints allow for much larger vertical shear forces to be transmitted between panels, however they are not preferred by contractors as they are significantly more expensive and slow the floor-to-floor construction cycle, which in some circumstances can effectively eliminate the cost advantage of adopting a precast concrete building core over a traditional cast in-situ core.

Figure 1. Example of a wet joint in jointed precast concrete cores.

The first objective of this experimental study was to assess the behaviour of WSP connections. Despite their widespread adoption in industry, little research has been performed into what their actual experimental capacity is and whether the connection has enough in-plane stiffness to allow composite behaviour to be developed. The second objective of this experimental study was to develop two new prototype connections for jointed precast building cores that did not require site welding or wet joints. The site welding component of WSP connections is also costly as it requires an additional trade to be contracted on site that would otherwise not be required while the major RC construction stage of a typical RC building project is happening.

2 Experimental Test Program

The experimental test program consisted of three test specimens denoted J01, J02 and J03. Each specimen consisted of two rectangular precast panels that were joined together to form the corner segment of a jointed precast building core, as illustrated in Figure 2. The first specimen (i.e. J01) was the baseline specimen and had 'welded stitch plate' (WSP) connections joining the panels together. The second specimen (i.e. J02) was the first prototype connection specimen and had 'grouted panel pocket' (GPP) connections joining the panels together. The third specimen (i.e. J03) was the second prototype connection specimen and had a 'post tensioned corbel' (PTC) connection joining the panels together. The connection details and overall specimens are presented in Figures 3 and 4 respectively.



Note: the system level tests shown in this figure are further described in Menegon et al. (2017a).

Figure 2. Testing methodology – system vs. component level test specimens.

The GPP connection consisted of one panel with 80x300 mm voids along the vertical end of the panel and the second with M20 cast-in ferrules along one side edge. When the two panels are erected adjacent to one another, the panel voids on the first panel line up with the cast-in ferrules along the side edge of the second panel. High tensile grade 8.8 M20 bolts are then inserted through the panel voids and into the ferrules. The void is then boxed up and filled with high strength cementitious grout. Two different methods are being proposed for grouting the pockets. The first method involves fixing a rectangular piece of formply across the void on the outside of the panel and then fixing a 'U' shaped piece of formply between the 20 mm panel gap. Both pieces of formply are fixed using nominal concrete screws (e.g. M6 x 50 mm long Ramset Anka Screws). The void is then filled with cementitious grout by pumping it through a pipe inserted between the panel gap and through the top of the 'U' section of formply fixed between the void. The second method is to firstly seal the panel gap around the void using gap filler and then fix a rectangular piece of formply with an outlet hole across the back of the void. Grout is then pumped through the outlet hole in the formply. Gap filler is a widely used product in the precast industry and is commonly used to fill the gaps between starter bar shutters. The first method was used in this study for constructing test specimen J02.

The PTC connection consisted of one panel that was the bottom section of the corbel and had a 26.5 mm nominal diameter Macalloy bar cast into it. The second panel was the top section of the corbel and had a 60 mm diameter corrugated grout tube cast into it. When erected, the Macalloy bar in the bottom section slotted through the corrugated grout tube in the top section. High early strength grout was used to dry pack the 20 mm horizontal gap between the top and bottom corbel sections. The day after the panel gap was dry packed with grout, the Macalloy bar was post tensioned. The grout tube was pumped full using a high strength cementitious grout immediately following the post tensioning of the Macalloy bar. The grout was pumped through a 15 mm diameter tube cast-in at the bottom of the grout tube. A second 15 mm diameter tube was cast-in at the top of the grout tube to give an indication of when the grout tube was full. Aitken Freemans Tecgrout HS was used for the grouting on both J02 and J03. The grout had a compressive cube strength of 70 MPa on test day.

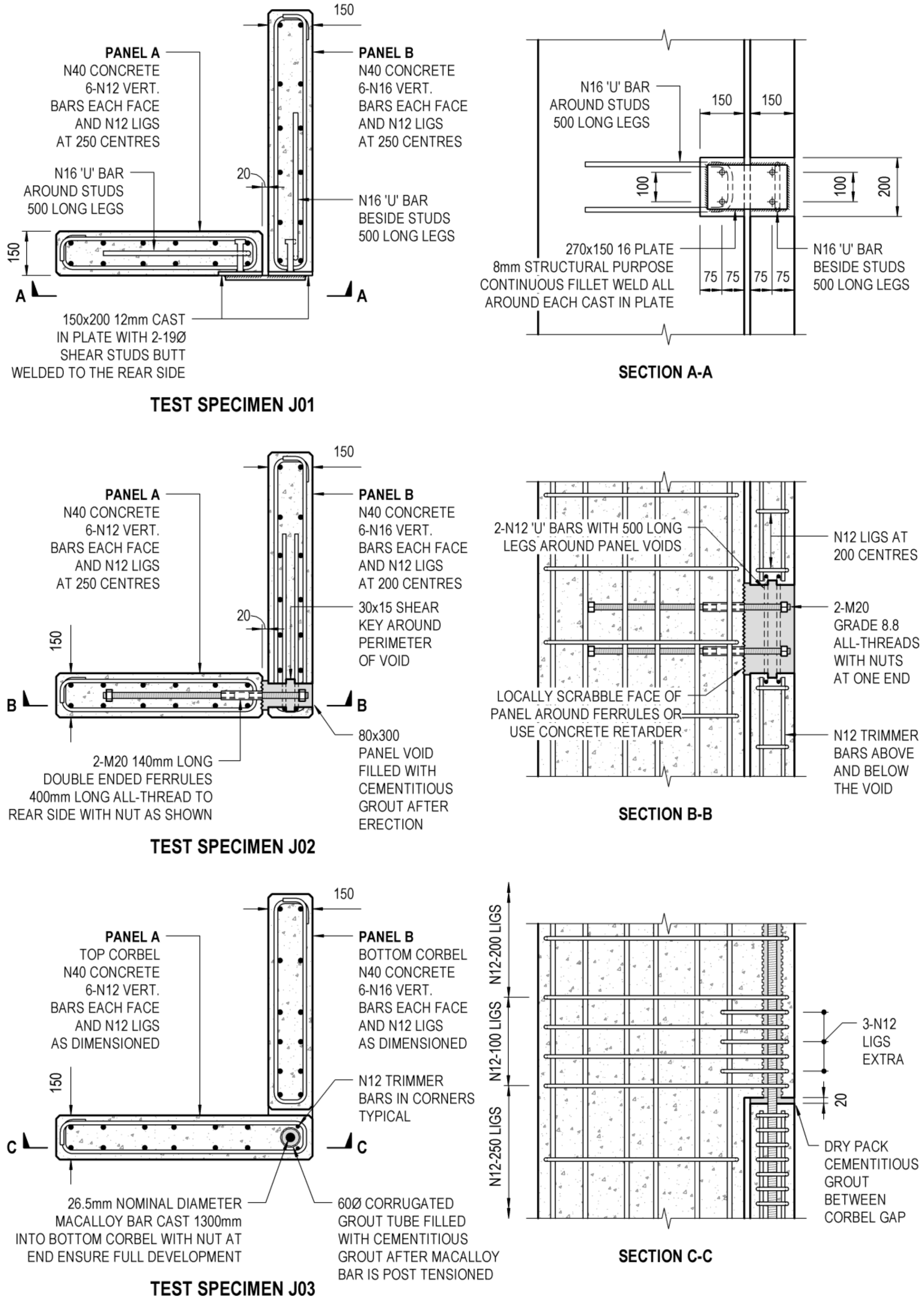


Figure 3. Test specimen connection details.

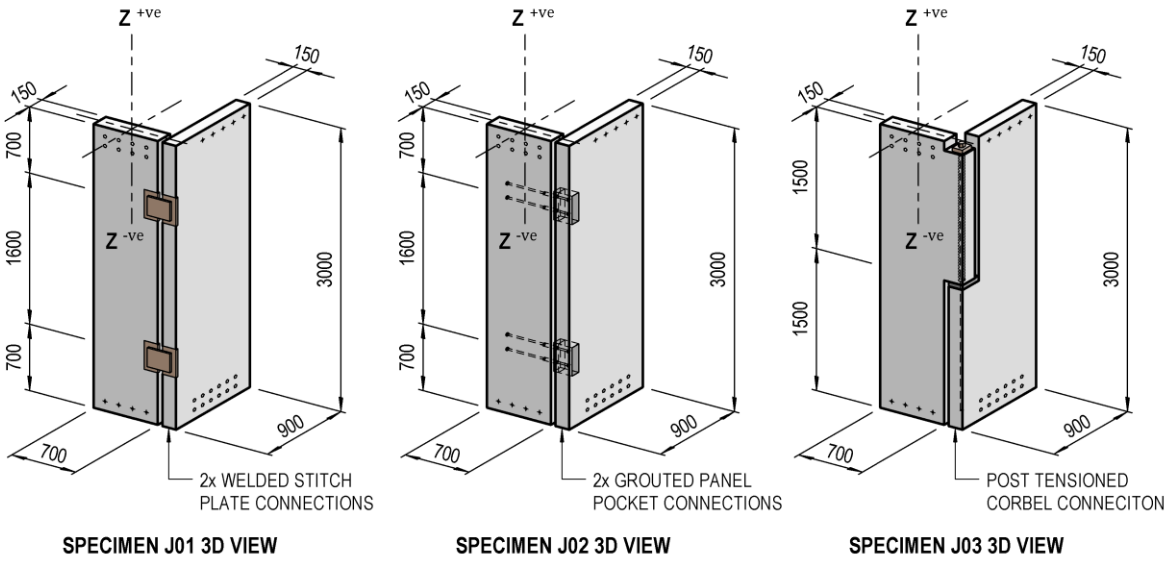


Figure 4. Test specimen 3D views.

3 Experimental Test Setup

The specimens were tested in the Multi-Axis Substructure Testing (MAST) System at Swinburne University of Technology. The MAST System is a state-of-the-art testing machine capable of applying six degree-of-freedom (DOF) loading to a test specimen in mixed-mode, switched-mode, hybrid simulation or a combination therein (Al-Mahaidi et al. 2018; Hashemi et al. 2015).

A series of structural steel plates and loading brackets were custom fabricated to connect the specimens to the MAST System and strong floor, while allowing the appropriate boundary element constraints to be present during testing. The top of panel A for each respective test specimen was connected to the crosshead of the MAST System using two fully fixed connection brackets, one either side of the panel. The specimens were loaded vertically through these two brackets. The bottom of panel B for each respective panel was connected to structural steel support plates using two fully fixed connection brackets, one either side of the panel. The structural steel support plates were in turn fully fixed to the strong floor, providing the base restraint for the specimens. These two respective connections were the primary load and restraint points for the test setup.

The bottom of panel A and the top of panel B for each respective test specimen had two connection brackets, one either side of the panel, which had vertical slotted holes. These brackets allowed the panels to move vertically between them, however they restrained their respective panel from moving laterally out-of-plane in either plan direction or twisting about the z-axis while the specimens were being loaded. This combination of fixed and vertically slotted connection brackets allowed the panel connections (i.e. WSP, GPP or PTC) to be subjected to purely vertical shear forces, similar to what would be observed in a system level response, without any out-of-plane forces developing due to the eccentric nature of the test setup. The test setup and loading brackets are shown and further described in Figure 5.

The specimens were loaded vertically in the z-axis of the MAST System, which is shown relative to the test specimens in Figure 4. The remaining five out-of-plane DOFs of the MAST System were commanded in displacement controlled mode to either zero movement or zero rotation for the duration of the test. The z-axis loading was applied in loading series that comprised two positive and two negative loading cycles. For test specimens J01 and J02, the positive and negative loading for each series was the same because both connections have a symmetrical response. J03 however, has an unsymmetrical response because under z-axis positive loading the corbel is pulled in tension and the Macalloy bar resists the load, and under z-axis negative loading the corbel is pushed in compression and a strut and tie mechanism resists the load. This meant J03 had an unsymmetrical loading protocol.

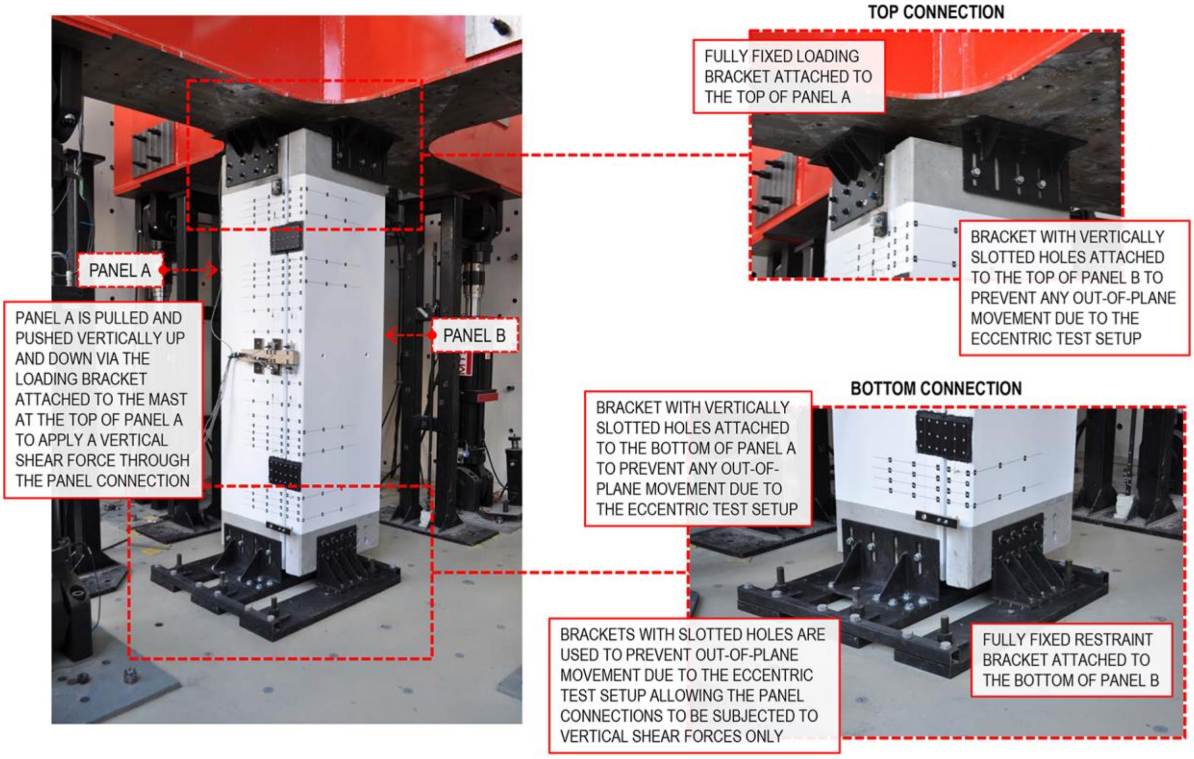


Figure 5. Test setup in the MAST System.

4 Preliminary Test Results

Test specimen J01 (i.e. WSP connections) had a maximum strength of 176 and 177 kN in the positive and negative loading directions respectively. The WSP connections exhibited a reasonably ductile failure mode (Figure 6), allowing a fair amount of connection deformation and gradual decline in strength before complete failure occurred, which was due to fracturing of the shear studs.

Test specimen J02 (i.e. GPP connections) had a maximum strength of 304 and 308 kN in the positive and negative loading directions respectively. The GPP connections exhibited a reasonably ductile failure mode (Figure 6), also allowing a fair amount of connection deformation after the maximum strength was exceeded. However, unlike J01, the strength declined more significantly between the respective cycles in the same loading series increment. The failure of the connection was governed by a cyclic degradation of the grouted pocket and shear failure of the bolts.

Test specimen J03 (i.e. PTC connection) had a maximum strength of 619 and 1061 kN in the positive and negative loading directions respectively. The PTC connection exhibited a stable hysteresis response (Figure 6) and was able to develop significant levels of ductility in the positive direction (i.e. when the corbel was ‘pulled up’ in tension), while it had a very sudden failure in the negative direction (i.e. when the corbel was ‘pushed down’ in compression). The ductile response in the positive direction was due to the response being governed by the Macalloy bar being pulled in tension and exhibiting inelastic strain hardening. The brittle response in the negative direction was due to general degrading and failure of the grout pad at the corbel interface. This is not believed to be due to the grout’s compressive strength being exceeded, but rather from cyclic degradation of the grout pad caused by yield penetration of the Macalloy bar being pulled in tension, which then induces transverse tensile stresses and vertical cracking in the grout. The maximum compressive stress in the grout at the corbel interface was conservatively (i.e. the Macalloy bar was assumed to take zero compression load, which would be false) calculated to be approximately 48 MPa, when the compressive load of 1061 kN was being applied. This is significantly less than the compressive strength of the grout, which was determined to be 70 MPa.

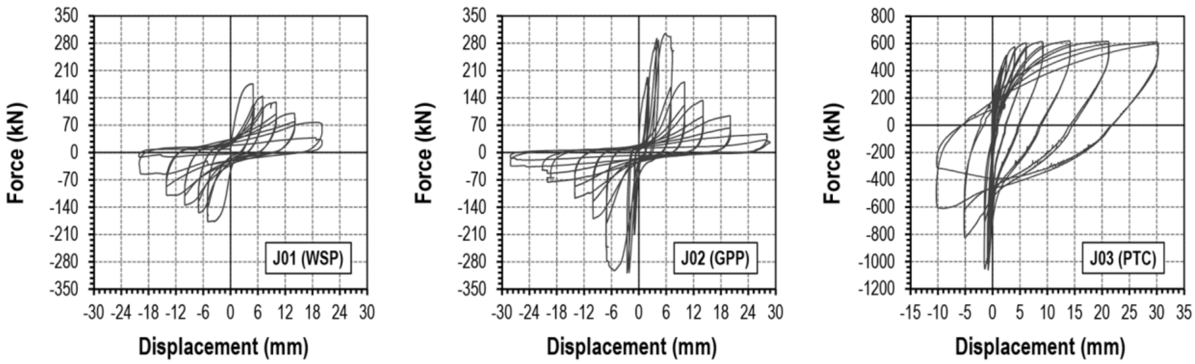


Figure 6. Force-displacement response for test specimens J01 (left), J02 (middle) and J03 (right).

The load distribution between the top and bottom WSP and GPP connections in test specimens J01 and J02 respectively was assessed by investigating the local movement of each connection. The movement of each respective connection, in each specimen, was measured using individual string potentiometers connected above and below each connection. The movement of the top and bottom connection in each specimen was approximately equal for the initial few load cycles where the maximum load was developed. The respective displacement of each connection started to differ after the maximum strength was reached and strength degradation started to occur. As such, it was deemed appropriate to assume the load was distributed evenly across both connections as the elastic response of the connection (i.e. the performance of the connection up to the maximum strength being reached) is the primary region of interest for these connections. While Figure 6 depicts the overall force-displacement response of test specimens J01 and J02, the connection stiffness (as discussed in the subsequent section) was assessed using the averaged response of the individual connections for each respective test specimen.

5 Connection Stiffness

The stiffness of each of the respective connections was determined by taking a secant stiffness from the origin to a point corresponding to 70% of the maximum strength of the connection being reached. A value of 70% was selected because this commonly represents approximately 80% of the design capacity (e.g. the capacity reduction factor for a shear stud in accordance with AS 2327.1 (Standards Australia 2003) is 0.85 and $0.85 \times 0.8 \sim 0.7$). The stiffness was calculated for both the positive and negative direction. The positive and negative direction stiffness values were averaged together and taken as the stiffness for each respective connection type. The stiffness for each connection is summarised in Table 1. The process for determining the stiffness for each connection is illustrated in Figure 7. It should be noted that Figure 7 shows the *connection* force-displacement response for test specimens J01 and J02, whereas Figure 6 shows the *overall* specimen response.

Table 1. Vertical connection stiffness.

Specimen	k^{+ve}	k^{-ve}	$k_{average}$
J01 (WSP)	36.40 kN/mm	52.54 kN/mm	44.47 kN/mm
J02 (GPP)	48.20 kN/mm	104.4 kN/mm	76.29 kN/mm
J03* (PTC)	1565 kN/mm	1938 kN/mm	1751 kN/mm

* The positive direction stiffness for test specimen J03 is taken as the initial stiffness of the connection prior to the post tensioning force being developed. After the initial post tensioning force of 200 kN was exceeded, the stiffness dropped to 120.9 kN/mm.

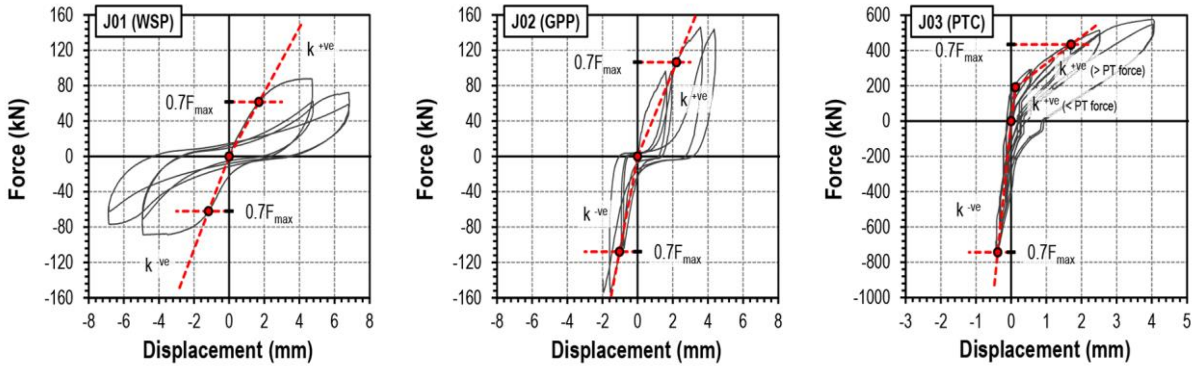


Figure 7. Vertical connection stiffness of test specimens J01 (left), J02 (middle) and J03 (right).

The WSP connection had the smallest stiffness of the three connections and was equal to 44.5 kN/mm. System level testing of a building core specimen with the same connection details as J01 (as shown in Figure 2 and discussed further in Menegon et al. (2017a)) suggested that this connection was not 'stiff enough' to allow effective composite action to be developed and resulted in a significant reduction in stiffness. This suggests that a stiffness of 44.5 kN/mm is too flexible for meaningful composite action to be developed.

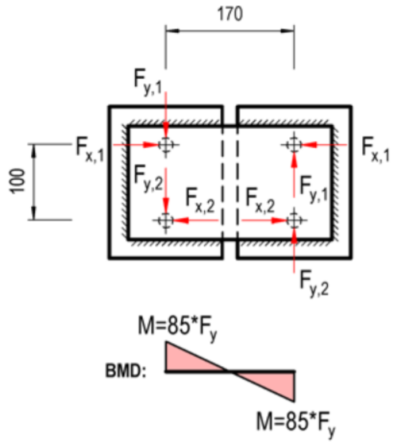
The first prototype connection, i.e. the GPP, was 73% stiffer than the WSP connection and had a stiffness of 76.3 kN/mm. The GPP connection also had a maximum capacity that was 73% higher than the WSP connection. This suggests the GPP connection is a superior alternative to WSP connections, while providing the additional benefit of avoiding site welding. Additional analysis is still required to assess if the higher stiffness of 76.3 kN/mm is sufficient to allow for more effective composite action to be developed. This is an ongoing area of research.

The second prototype connection, i.e. the PTC, was nearly 40 times stiffer than the WSP connection, with a stiffness of 1750 kN/mm. The very high stiffness and strength of the PTC connection suggest it would be close-to or equally as effective as a wet joint in precast building cores, which essentially provides the equivalent performance of a cast in-situ element. It should be noted that once the PTC is loaded in tension to a force exceeding the initial post tensioning force, which was 200 kN in test specimen J03, the stiffness significantly reduces to 121 kN/mm. It is recommended in a practical application that the post tensioning force that is applied should be greater than the required design shear force the connection is required to resist. It should be noted that the post tensioning force applied to test specimen J03 was limited due to construction constraints in the laboratory.

6 Blind Prediction Study

A blind study assessing the strength of the WSP connection used in test specimen J01 was undertaken with multiple structural engineering design consultancies in Australia. The blind study highlighted that there was a wide variation of opinion to what the capacity of this connection was. The majority of designers thought the shear studs would be the critical component of the connection, however amongst them, there was considerable variation regarding how they calculated the loading on the shear studs. Typically, they fell into three groups: the first group assumed a moment was developed locally across the connection equal to the vertical shear force applied to the connection multiplied by half the distance between the shear studs on each adjacent panel (i.e. a double curvature moment gradient was assumed across the stitch plate); the second group also assumed a moment was developed locally across the connection, however they said it was equal to the vertical shear force multiplied by the total distance between the shear studs on each adjacent panel (i.e. a single curvature moment gradient was assumed across the stitch plate); and the third group assumed no moment was developed locally across the connection. As such, each of these groups had considerably different loading scenarios and design actions for the shear studs. The three different loading scenarios result in different connection strengths as follows: the group one approach results in the maximum shear capacity of the connection being equal to 1.01 times the shear capacity of a single shear stud (as shown in Figure 7); the group two approach results in the maximum shear capacity of the connection being equal to 0.56 times the shear capacity of a single shear stud; and the group three approach results in the maximum shear capacity of the connection being equal to 2 times the shear capacity of a single shear stud. This meant the submissions for the strength of the connection, where it was assumed the shear studs were the critical component, ranged from about 52 to 186 kN (assuming a capacity reduction factor of 1.0). While the majority of participants thought the shear studs would be critical, there were also submissions that thought the weld strength would be the critical component and in this instance, the capacity was thought to be about 230 kN.

The failure mechanism for the WSP connections in test specimen J01 was failure of the shear studs and the maximum strength was 88 kN per connection. The WSP connection had two 19 mm diameter shear studs on each cast-in plate, which have a maximum capacity of 93 kN in accordance with AS 2327.1. Using the free body diagram and associated shear stud load distribution in Figure 8, which assumes a double curvature moment distribution across the stitch plate (as per the group one assumption), the maximum strength of the connection is 94 kN. The capacity reduction factor for shear studs in accordance with AS 2327.1 is 0.85, meaning the 'ultimate' shear strength of the connection is then 80 kN. This means the group one assumption discussed above is the most appropriate design model for the connection. Forty per cent of the participants assumed this design procedure, meaning more than half the participants from industry incorrectly calculated the capacity of this connection, despite it being a widely used 'industry standard' connection.



$$F_y = F_{y,1} + F_{y,2}$$
$$M = F_y \times \left(\frac{170}{2}\right) = 85F_y$$
$$F_{y,1} = 0.5F_y$$
$$F_{x,1} = \frac{M}{100} = \frac{85F_y}{100} = 0.85F_y$$
$$F_{r,1} = \sqrt{(F_{x,1})^2 + (F_{y,1})^2} = \sqrt{(0.85F_y)^2 + (0.5F_y)^2} = 0.99F_y$$
$$\therefore [F_y]_{max} = 1.01 \times [F_{r,1}]_{max}$$

Where: $[F_{r,1}]_{max}$ = maximum shear capacity of the shear stud

Figure 8. Free body diagram and bending moment diagram of stitch plate connection.

7 Conclusions

This paper provides the details and preliminary results of a recent experimental study into the performance of precast panel connections in jointed precast building cores. The experimental program included one welded stitch plate (WSP) connection, which is the industry standard precast building core connection in Australia, and two innovative new prototype connections. Both prototype connections were developed to replace WSP connections in an effort to eliminate the need for site welding. The first connection was developed as a direct alternative to WSP connections, whereas the second connection was developed as more of an alternative to wet joints, which are required when very large shear forces are needed to be transferred across panel joints.

The first new prototype connection (i.e. the GPP) had a maximum strength that was 73% greater than the 'baseline' industry standard WSP connection. Similarly, the stiffness was also 73% greater, hence providing proof of concept that the GPP connection is potentially a viable substitute for WSP connections in jointed precast building cores.

The second new prototype connection (i.e. the PTC) had a much greater maximum strength and more importantly, had a stiffness that was about 40 times greater than the WSP connection. Similarly providing proof of concept that the PTC is potentially a viable substitute for wet joints in jointed precast building cores.

A blind prediction study was also performed with multiple structural engineering design consultancies in Australia to assess the strength of the WSP connection used in this experimental study. The study showed that there is widespread opinion on how to calculate the capacity of these connections, which resulted in participants having submissions that varied by a factor of nearly five. Approximately 40% of the participants calculated the strength of the connection to within 10% of the actual failure load of the test specimen. However, other participants calculated the strength to be up to 45% weaker or 230% stronger than the actual capacity.

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9 References

Al-Mahaidi, R., Hashemi, J., Kalfat, R., Burnett, G. and Wilson, J., 2018. *Multi-axis Substructure Testing System for Hybrid Simulation*, Springer Nature, Singapore.

Hashemi, M.J., Al-Mahaidi, R., Kalfat, R. and Burnett, G., 2015. Development and validation of multi-axis substructure testing system for full-scale experiments, *Australian Journal of Structural Engineering*, Vol. 16(4), pp. 302-315.

Menegon, S.J., Wilson, J.L., Lam, N.T.K. and Gad, E.F., 2017a. Comprehensive Experimental Testing Program of Limited Ductile Reinforced Concrete Walls, *Proceedings of the Australian Earthquake Engineering Society 2017 Conference, November 24-26*, Canbeera, ACT.

Menegon, S.J., Wilson, J.L., Lam, N.T.K. and Gad, E.F., 2017b. RC Walls in Australia: Reconnaissance Survery of Industry and Literature Review of Experimental Testing, *Australian Journal of Structural Engineering*, Vol. 18(1), pp. 24-40.

Standards Australia, 2003. *AS 2327.1-2003 Composite Structures, Part 1: Simply supported beams*, Standards Australia, Sydney, NSW.