
Improving seismic performance of panel-to-foundation connections in low-rise precast concrete buildings

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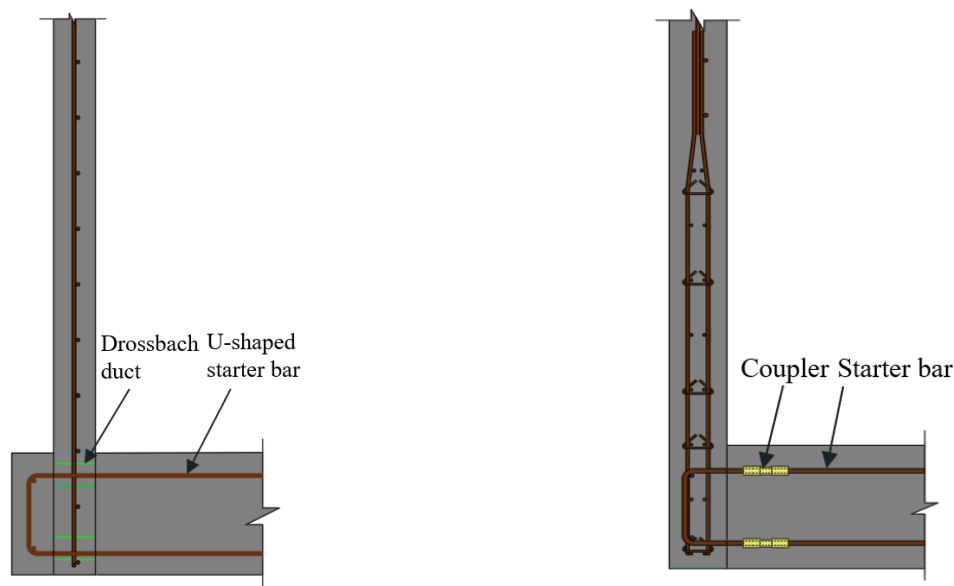
ABSTRACT

Low-rise precast concrete buildings are a common building stock in New Zealand. Post-earthquake reconnaissance and experimental studies have identified vulnerabilities in dowel-type panel-to-foundation connections that use threaded inserts. These connections lack a robust load path when subjected to out-of-plane loading, resulting in brittle joint failure and the inability for the panel to resist in-plane loads. Two alternative connection details have been proposed that provide robust load paths during seismic loading whilst maintaining constructability. Two specimens were subjected to in-plane loading, preceded by a single out-of-plane cycle applied first to induce dowel action in the foundation starter bars. The results show that the foundation details performed effectively when subjected to in-plane loading, limiting damage within the joint region and providing a robust connection between the panel and foundation. Both details developed the nominal flexural strength of the panel with a displacement capacity of 1.5% drift. The failure mechanisms of the panels were similar for both details, characterized by concrete spalling, reinforcement buckling, and bar fracture prior to failure. The in-plane behaviour of the panels indicated that flexural deformation contributed most to the total deformation, while shear deformation, sliding, and strain penetration accounted for only a small proportion.

1 INTRODUCTION

Precast concrete wall buildings are a common low-rise building type in New Zealand. For precast low-rise industrial and commercial buildings, dowel-type panel-to-foundation connections are predominantly used (Beattie, 2007). The 2010/2011 Canterbury earthquakes highlighted vulnerabilities and concerns associated with dowel-type connections employing threaded inserts (SESOC, 2013). Hogan et al. (2018) demonstrated that such dowel-type panel-to-foundation connections lack a reliable load path when subjected to joint opening rotations and the connections exhibit brittle failure modes. The study also proposed three alternative connection details: i) increasing the embedment depth of the threaded insert to the back of the panel; ii) providing an additional reinforcing link bar in the joint region; and iii) providing confinement stirrups in the

joint region. These details were experimentally shown to achieve the nominal flexural capacity and maintain joint integrity. However, they did not meet the durability requirements specified in NZS3101:2006 (2017), making them unsuitable for common construction practices. To address the problem and align with industry recommendations, Han (2020) proposed two new connection details: one using horizontal starter bar connections with Drossbach ducts and a singly reinforced panel (Figure 1(a)), and another using coupler bar connections with a doubly reinforced panel (Figure 1(b)). These two connection alternatives accommodate both panels that are located both on and away from property boundary. Out-of-plane (OOP) tests conducted on the foundation connections demonstrated that no cracks were observed in the joint region for either connection, except for a single horizontal crack in the panel below the foundation level in the doubly reinforced panel details, indicating the reliable connection between the panel and the foundation. Panels in both details were able to achieve their nominal moment capacity up to 6% drift. Despite their ability to maintain joint integrity and provide a reliable load path in out-of-plane loading, it is necessary to investigate their behaviour when subject to in-plane (IP) loading. Therefore, two experimental tests, one investigating each of the connection details shown in Figure 1, were conducted to validate the proposed foundation details. Their behaviour was examined in terms of damage patterns, hysteresis response, failure modes, deformation components, and curvature distribution.



(a) Singly reinforced panel base details

(b) Doubly reinforced panel base details

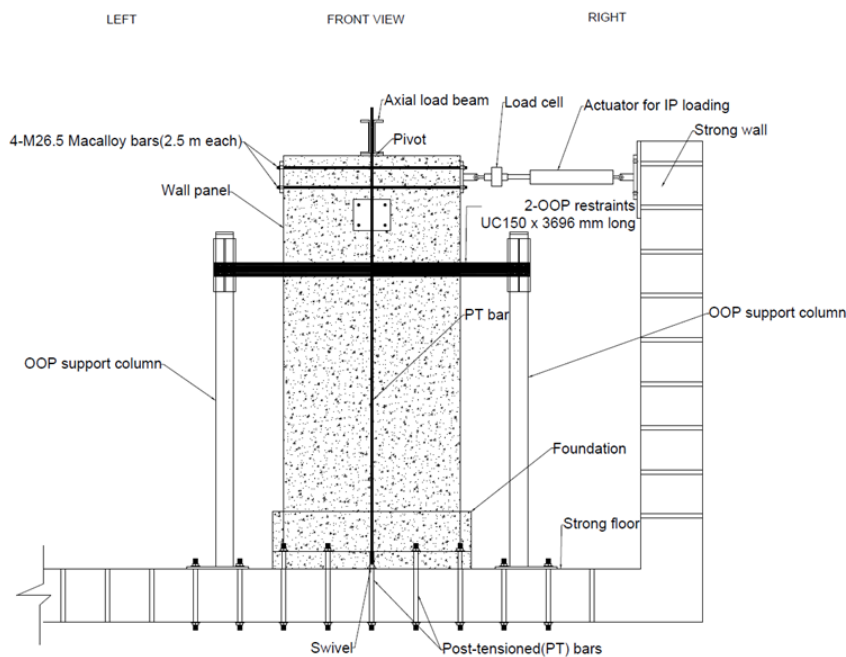
Figure 1: Schematic representation of proposed panel-to-foundation dowel connections

2 SPECIMEN AND LOADING DETAILS

The precast panels utilized in the testing represented a 10 m high wall in low-rise precast industrial buildings. In these buildings, lateral resistance is provided by steel portal frames in the transverse direction and by precast panels in the longitudinal direction, supporting a light metal roof. These buildings are generally designed for a nominally ductile seismic response. Consequently, the precast panels are often detailed with a single layer of centrally located reinforcement, with ratios kept close to the minimum reinforcement values specified in NZS3101:2006(2017) (Seifi et al., 2016). Two panel configurations were examined: a singly reinforced panel (150 mm thick with a longitudinal reinforcement ratio of 0.30%) and a doubly reinforced panel (175 mm thick with a longitudinal reinforcement ratio of 0.36% at the base). Both panels were

4500 mm high and 2000 mm long. Details of the reinforcement and panel-to-foundation connections are provided in Han (2020).

The panels were initially subjected to one OOP loading cycle at 1% drift to separate the panel from the foundation and induce dowel action in the starter bars (Han, 2020). This OOP cycle was followed by IP loading with a constant axial load of 0.34% to represent the self-weight of the remaining portion of a prototypical 10 m tall wall. The consecutive IP drift levels were 0.12%, 0.20 %, 0.30%, 0.5%, 0.75%, 1.0%, 1.5%, 2.00% and 2.50%, applied in both positive and negative directions. Three cycles were applied at each drift. The test set-up used in the experiment is shown in Figure 2. The OOP and IP loads were applied with an actuator at the top of the wall reacting respectively against a steel column and a strong wall. Two OOP restraint frames were provided to prevent the OOP instability during the IP testing as illustrated in Figure 2. Figure 3 shows the instrumentation used during the testing. Draw-wire displacement transducers (DW) were used to measure lateral displacement at different heights of the specimen, while LVDTs measured the uplift of the foundation relative to the strong floor. Deformation and strain at the panel base were captured by portal gauges (PG), and panel shear deformation was measured using diagonal linear potentiometer gauges (LG).



(a) Schematic drawing of the set-up



(b) Photograph of the experimental set-up

Figure 2: Test set-up

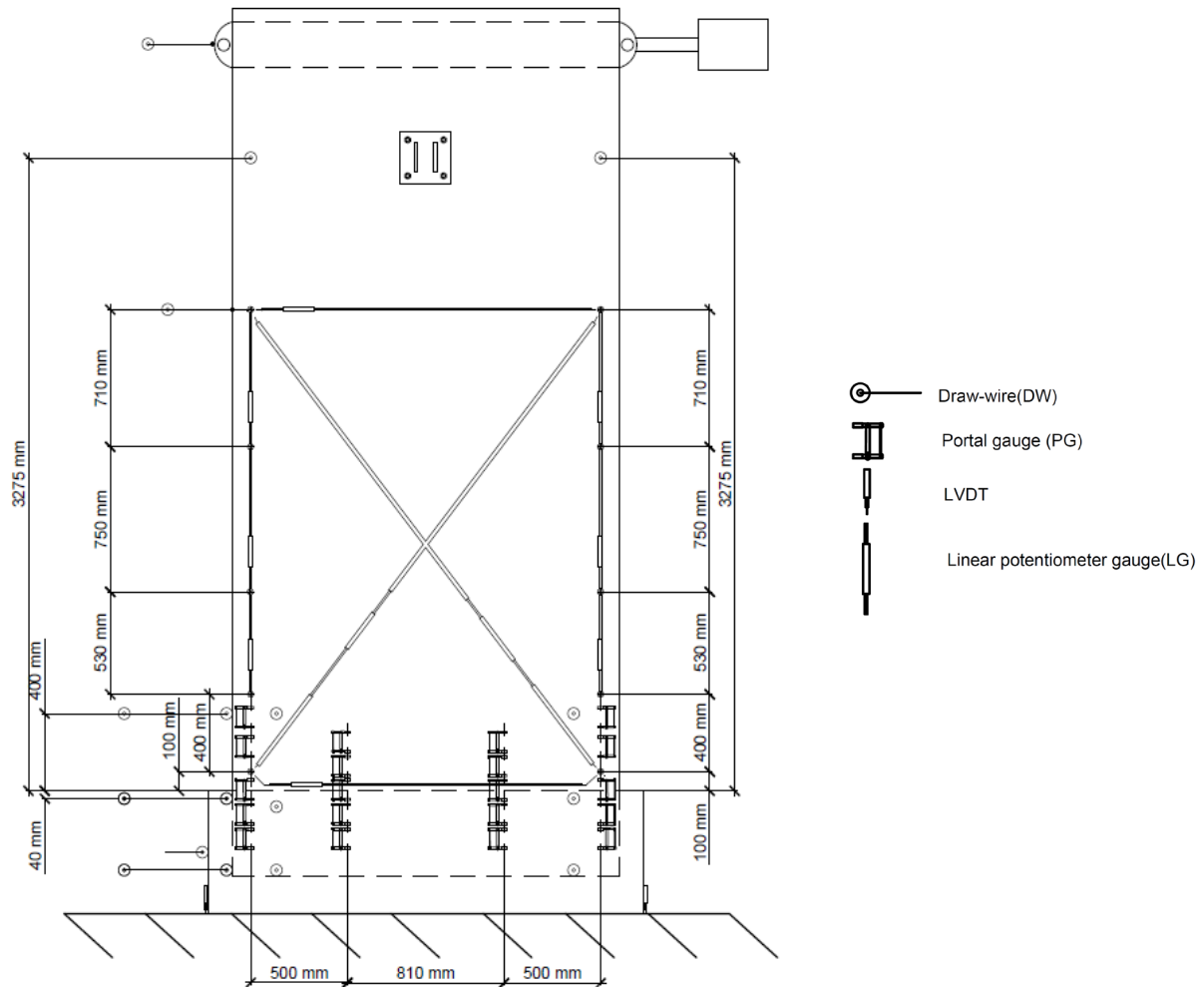


Figure 3: Instrumentation used in the test

3 RESULTS AND DISCUSSION

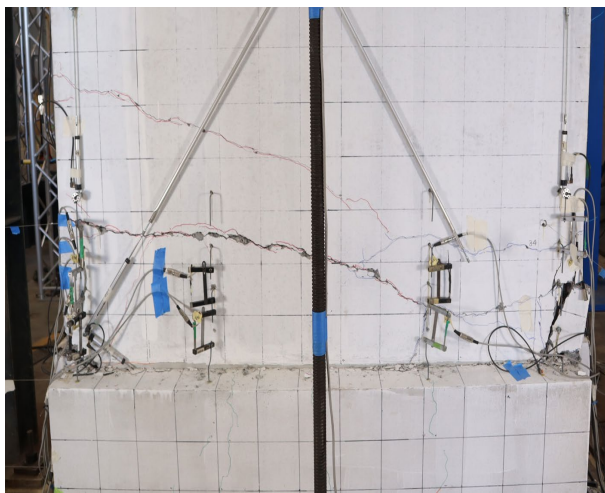
3.1 Damage patterns and hysteresis response

3.1.1 Singly reinforced panel base details

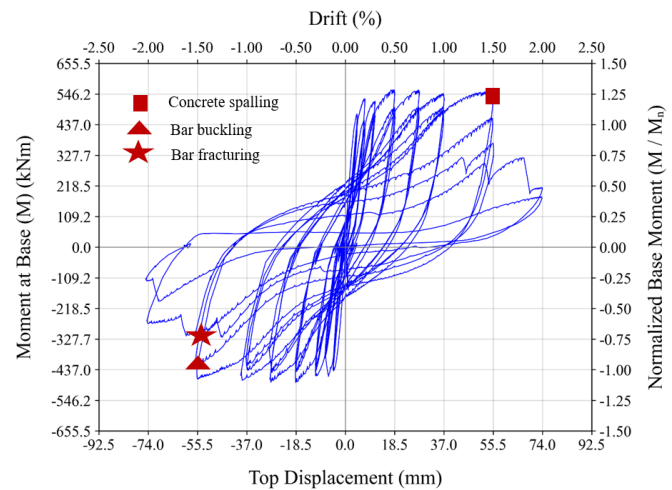
The specimen was subjected to the IP drift up to 2.00% drift. During the OOP cycle at 1% drift, no separation between the internal foundation and the panel was observed. However, vertical cracks of approximately 0.25 mm developed along the exterior foundation-panel joint, causing separation at the interface. No panel sliding was observed at either panel-foundation interface during IP loading and no new cracks formed at the interfaces. The connection region remained largely undamaged, with minor surface spalling observed in the edges of the panel around the top of foundation, which was not significant enough to affect panel behaviour.

Two large flexural cracks were observed near the panel base above the foundation level, as depicted in Figure 4(a). Figure 4(b) shows the stable hysteretic response of the panel, where moment at base (M) refers to the moment at the top of the foundation, the normalized base moment is defined as the ratio of the base moment (M) to the nominal flexural strength (M_n) of the base section. Concrete spalling occurred at +1.5%

drift during the first cycle, resulting in approximately 20% reduction in panel strength. This concrete spalling reduced confinement, leading to bar buckling at -1.5% drift in the second cycle and bar fracturing during the third cycle at -1.5% drift. In each drift level, the strength and stiffness decreased after the first cycle. The wall panel was able to develop its nominal moment capacity up to the second cycle at 1.5% drift.



(a) Damage at 1.5% drift



(b) Moment-drift response

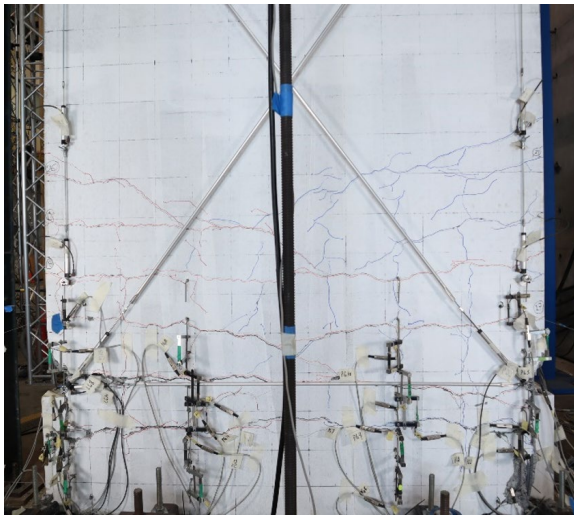
Figure 4: Damage and hysteresis response of singly reinforced panel base details

3.2.2 Doubly reinforced panel base details

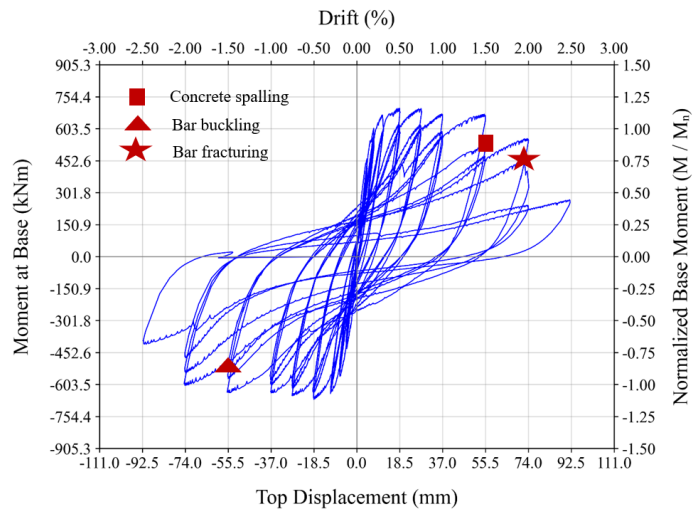
The specimen was tested up to 2.5% drift. Apart from a small separation (0.05 mm) at the top of panel-foundation interface, no other damage was observed in the joint region during the 1% OOP drift. Following an IP drift of 0.2%, a horizontal flexural crack (0.4 mm) developed in the panel below the foundation level, near the top starter bar location, as shown in Figure 5(a). The crack width increased with increasing drift levels but did not significantly affect the ability of the panel to develop its nominal moment capacity. No new panel-foundation interface cracks formed during IP loading, and damage in the joint region was limited to concrete spalling at both the left and right edges of the panel near the top of the joint region.

Two dominant horizontal cracks, one above the foundation and another at the top starter bar location, developed near the panel base, with other cracks in the bottom 1.5 m of the panel above the foundation as can be seen in Figure 5(a). The behaviour was similar to that of singly reinforced panel base details in terms of strength and stiffness degradation, which can be illustrated by the stable hysteresis response shown in Figure 5(b). When the drift level reached +1.5% during the second cycle, concrete spalled and panel strength dropped by 22%, followed by bar buckling at -1.5% drift during third cycle and bar rupturing during second cycle at +2% drift. The panel reached its nominal moment capacity up to the first cycle at 1.5% drift.

The doubly reinforced panel exhibited more distributed cracking compared to the singly reinforced panel. This behaviour was attributed to the higher ratio of nominal flexural strength to cracking moment (M_n/M_{cr}) in the doubly reinforced panel (1.35) compared to the singly reinforced panel (1.13). However, failure in both panels was primarily governed by two large cracks at the panel base, similar to observations in tests on concrete shear walls with minimum vertical reinforcement by Lu et al. (2017).



(a) Damage at 1.5% drift



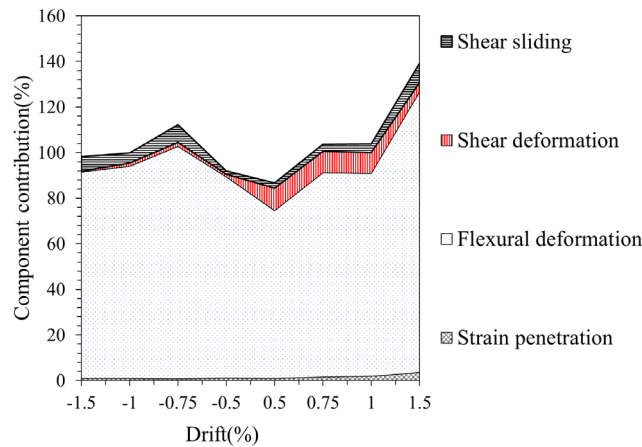
(b) Moment-drift response

Figure 5: Damage and hysteresis response of doubly reinforced panel base details

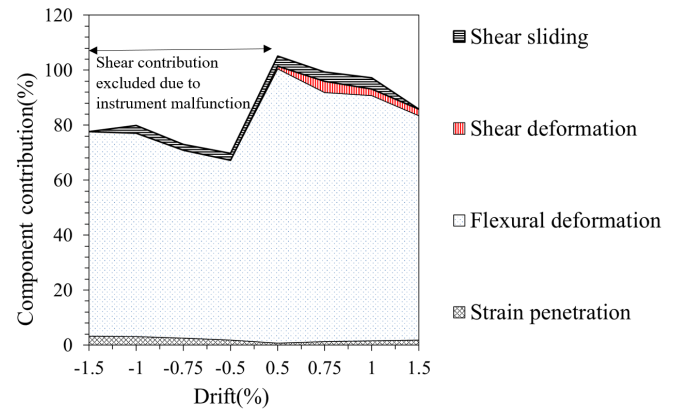
3.2 Panel deformation components

The contribution of individual deformation components to the total panel displacement was investigated during the first cycle at each in-plane (IP) drift level and is shown in Figure 6. Flexural deformation, shear deformation, strain penetration, and shear sliding at the panel–foundation interface were included in the study. Flexural deformations were determined by double-integrating the curvatures derived from average strains calculated from vertical distance measurements obtained using portal gauges and linear potentiometers along the wall edges, assuming plane sections remain plane. Shear deformations were calculated from diagonal displacements recorded by diagonally placed linear potentiometers and corrected as recommended by Hiraishi (1984) and Shegay et al. (2018). Strain penetration was computed by dividing the difference in vertical displacements recorded by portal gauges attached to the wall base and the foundation by the horizontal distance between them. Shear sliding at the panel–foundation joint was measured as the difference between horizontal displacement recorded at the wall panel base and that recorded at the wall panel at top of the foundation.

In both panels, flexural deformations contributed the most to the total displacements, whereas the shear displacements accounted for up to 9% of the total deformation. This low shear contribution was primarily attributed to the panel shear span ratio of 2 and was consistent with the finding by Lu et al. (2017) for lightly reinforced walls tested with the same ratio. There was no significant rocking (strain penetration) in either panel during the test, as it represented at most 3% of the total displacement. Similarly, shear sliding at the joint was observed to be small, ranging from negligible to 6% in terms of total wall displacement as drift levels increased. The deformation components aligned with the crack patterns observed in the panels. As illustrated in Figure 4(a) and Figure 5(a), flexural cracks were dominant in both panels, consistent with the calculated deformation response. The observed strain penetration cracks at the panel–foundation interface were considerably smaller than the cracks within the panel for both connection details. However, strain penetration in the doubly reinforced panel was slightly higher than that in singly reinforced panel, which may be attributed to the presence of foundations on both sides of the singly reinforced panel. Similarly, the cracks within the panel were significantly larger than the cracks associated with shear sliding in the joint region for both connection details.



(a) Singly reinforced panel

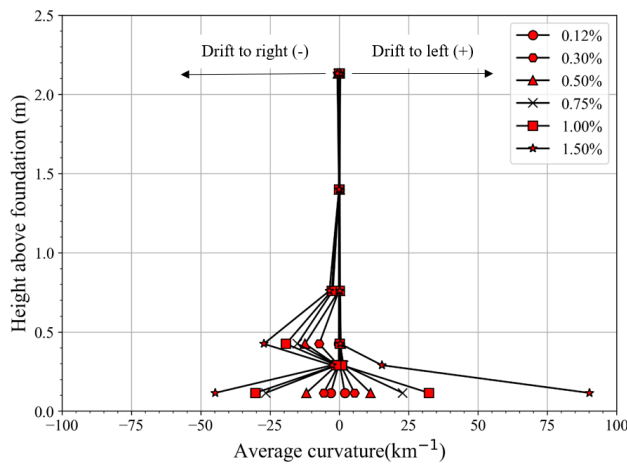


(b) Doubly reinforced panel

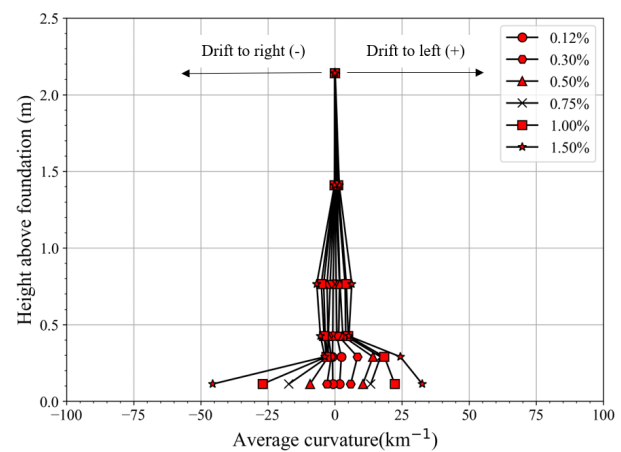
Figure 6: Deformation components of the panel

3.3 Curvature distribution

Curvatures along the height of the panel were computed from the average strains derived from the vertical displacement measured at both edges of the panel. The calculated curvatures were plotted up to approximately mid-height of the panel, where instruments were attached, and represent the peak values of the first cycle at each drift level. The average curvatures up the height of the wall panel are shown in Figure 7. The curvatures adjacent to the foundation also included the effect of strain penetration, resulting from the wall base rotation. The curvature profiles were consistent with the crack patterns observed in both panels, as shown in Figure 4(a) and Figure 5(a), and further confirmed the observed panel behaviour. The maximum curvatures for both panels occurred near the panel base, indicating that the peak flexural deformation occurred at the wall base. However, the curvature profiles were not distributed as assumed in the traditional curvature distribution, in which curvatures decrease along the wall height, due to non-uniform crack distribution along the panel height. As shown in Figure 7(a), the curvature at the height of 425 mm for singly reinforced panel was larger than that at 290 mm during the negative cycles, because of the wide crack that formed at 425 mm height, as illustrated in Figure 4(a). The curvatures were more distributed in the doubly reinforced panel than in the singly reinforced panel, as depicted in Figure 5(b).



(a) Singly reinforced panel



(b) Doubly reinforced panel

Figure 7: Average curvature distribution over the height of the panels

4 CONCLUSIONS

Experimental tests of two alternative panel-to-foundation connections were performed under IP loading, with one cycle of OOP loading. The following key conclusions are drawn from the test results:

- Both panel-to-foundation connections effectively controlled joint-region failure and ensured strong anchorage between the panels and the foundation. Either connection type can provide adequate load transfer for low-rise precast wall systems and allow the panels to emulate lightly reinforced cast in situ walls.
- Both specimens exhibited flexural cracking near the panel base, with a limited number of dominant cracks, followed by concrete spalling, reinforcement buckling, and fracture. Despite this damage, the hysteresis loops remained stable, with panels of both connection types developing their nominal flexural strength up to 1.5% drift without significant loss of lateral capacity. This performance verifies that the specimens achieved the necessary ductility to be classified as nominally ductile structures under the provisions of NZS3101:2006 (2017).
- Flexural deformation was the dominant contributor to total displacement, while shear deformation, shear sliding at the panel-foundation interface, and strain penetration contributed only marginally, indicating a flexure-dominated panel response.
- The curvature profiles aligned with the observed crack patterns, with peak curvatures at the wall base. This confirms that the failure initiated at the base, and the connection details restrained joint failure.

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