Preliminary test results of precast concrete panels with grouted connections

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ABSTRACT: Following the 2010/2011 Canterbury earthquakes the seismic design of buildings with precast concrete panels has received significant attention. Although this form of construction generally performed adequately in Christchurch, recommendations for the design of connections between precast concrete panels that were presented in the SESOC Interim Design Guidance were more stringent than had typically been adopted prior to the earthquakes. Recognising the value of having experimental evidence to support standard detailing for the connections between precast panels, a review was conducted to establish representative details used in both existing and new precast concrete buildings in New Zealand. The results of the panel detail review were used to develop an experimental test program to verify the seismic performance of panel-foundation connections when using a grouted metal duct. Various parameters such as the wall aspect ratio, different configurations of the SESOC recommended detailing, and the magnitude of axial load were included in the experiment program. The preliminary results of the experiments are reported.

1 INTRODUCTION

Since the 1960s there has been significant use of precast concrete in New Zealand for a variety of structural components, including floor systems, walls, moment resistant and gravity load frames and non-structural cladding panels (Crisafulli et al. 2002). The advantages of the precast concrete construction method are increased speed of construction, optimised material consumption, reductions in on-site labour work and improvements in quality control (PCI 2010). In particular, precast concrete panels have been widely used in New Zealand in building construction, from low-rise structures such as warehouses to high-rise buildings. The large strength and stiffness of concrete walls against both vertical gravity loads and horizontal wind and earthquake loads allow them to be used as a primary force resisting system.

Precast concrete walls display a mix of flexural, shear, rocking and base sliding deformation mechanisms when they are subjected to lateral loads, based on the connection detailing and their strength and ductility (Becker et al. 1980). Therefore connection detailing has an important influence on the load-deformation behaviour of the wall and consequently on overall seismic performance of precast concrete wall structures. In this study, details of precast panels and the associated connections used for connecting panels to foundation are reviewed. According to the collected panel details, an experiment programme was designed with detailing that represented the most commonly used detailing in New Zealand. The details of the experiment programme and the conducted tests are addressed in this research.

2 CURRENT WALL-TO-FOUNDATION CONNECTIONS

A review of manufactured precast concrete panels was undertaken in order to develop a comprehensive understanding of the common typologies for connections between precast concrete panels and foundations. The database of connection types was developed by searching through historical drawings from precast panel manufacturers and Auckland Council archives. A total of 108 projects in Auckland, Wellington and Christchurch that had been completed between 2003 and 2014 were reviewed. These projects involved the use of more than 4800 precast panels. The typologies of the detailing were dependent on the functionality of the precast wall. Based on their reinforcement content and connection details, precast concrete walls can be divided into three groups: (1) Multi-storey building walls, (2) Low-rise industrial building walls, and (3) retaining walls. The collected data represented 37 multi-storey buildings, 68 single storey warehouses and 3 retaining wall projects.

Different detailing is used to connect precast concrete walls to their foundations based on the type of structure and the magnitudes of the loads applied to them. These connection types are generally based on one or a combination of the following three categories: (1) dowel connections, (2) grouted connections, and (3) post-tensioned connections.

2.1 Dowel connections

In dowel connections, bars anchored inside the panel are extended further into the foundation in order to provide the structural connection. Dowel bars are usually anchored using different methods such as standard hooks (Figure 1a), threaded inserts (Figure 1b) or bolts (Figure 1c).

Dowel connections are simpler than other types of connections and do not need any extra equipment for grouting or post-tensioning. The main drawback of using dowel connections is the shallow embedment of the connection bars inside the wall which may lead to connection breakout. These characteristics of dowel connections allow them to be used in warehouses and low-rise buildings which are usually subjected to lower seismic loads due to their lower weight. In the conducted review it was found that dowel connections were used in more than 65 percent of wall-foundation connections.

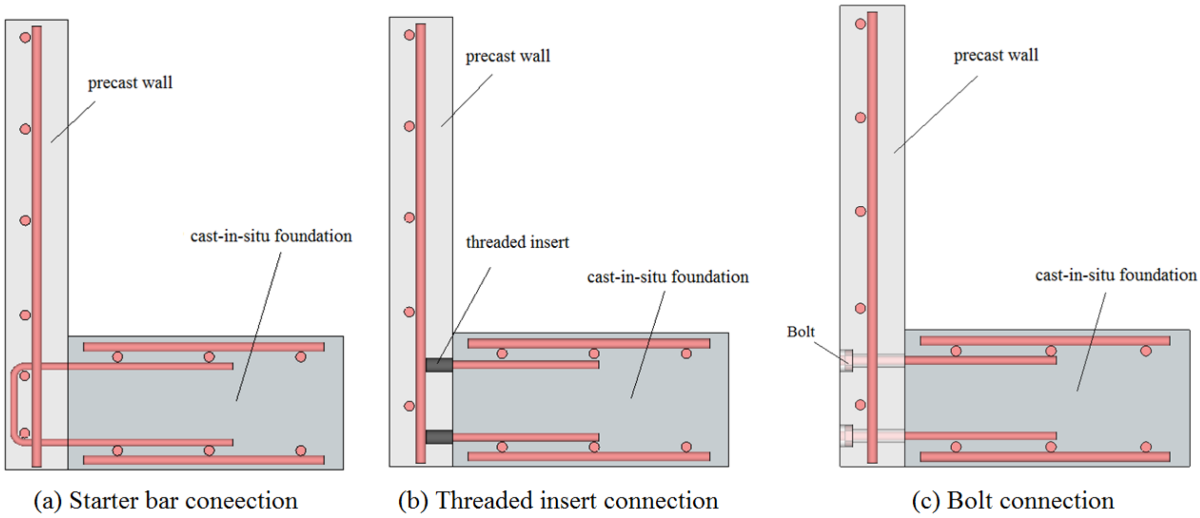


Figure 1. Side views of typical wall-foundation dowel connections.

2.2 Grouted connections

In the sampled data grouted connections were used in 32 percent of wall-foundation connections, which mostly were in multi-storey buildings. The straight force path in grouted connections decreases the possibility of brittle connection failure, thus for seismic design these connections are more appropriate than dowel connections. Two types of connections referred to as 'metal duct' and 'grouted sleeve' connections have been in use in New Zealand.

2.2.1 *Metal duct connections*

Metal ducts are usually made of corrugated alloy to improve bonding between ducts and concrete. Metal ducts are placed inside walls during casting and then connection bars from the foundation are placed inside the ducts during site installation, with the ducts then filled with non-shrink grout. An example of a metal duct connection is shown in Figure 2a. The gap between panels and the foundation is usually grouted.

A problem of grouted connections is loss of panel stiffness and strength due to the ducts causing a reduction of effective wall cross section. This problem weakens both the connecting bars and the transverse reinforcement anchorages due to spalling of concrete around the ducts when the wall is subjected to large cyclic loads. From 2011 onwards the Structural Engineering Society of New Zealand (SESOC 2013) has recommended the use of additional longitudinal bars and stirrups around the duct in order to provide concrete confinement to alleviate this problem. Confinement increases the strength and ductility of concrete and consequently prevents brittle connection failure. Another recommendation by SESOC is debonding of the connection reinforcement from the foundation concrete to prevent stress concentration in bars, as the debonded connection bars can undergo larger elongation before rupturing. Whilst this debonded detail may improve the behaviour of the connection, it has a number of problems in practice. Due to the limited space around the duct it is difficult to place stirrups in the low thickness walls which are commonly used in the New Zealand. Furthermore, the stirrups increase the amount of reinforcement around ducts, which might result in a decrease of concrete quality around ducts. In the sampled data, metal ducts were used in 65 percent of wall-foundation grouted connections.

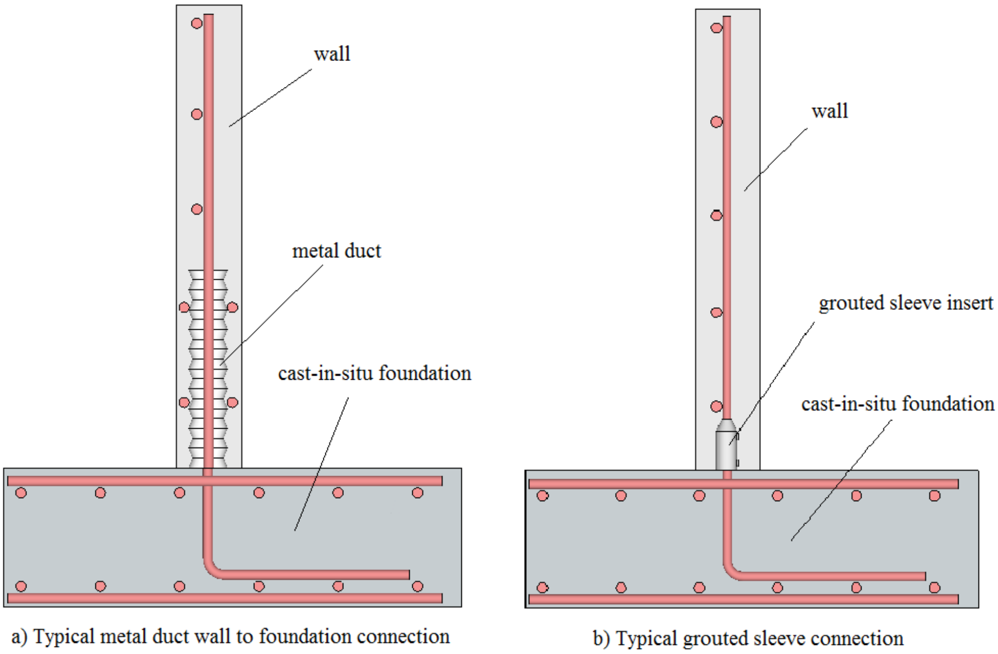


Figure 2. Side views of typical wall-foundation grouted connections.

2.2.2 *Grouted sleeve connections*

Another type of grouted connection is based on using grouted sleeve inserts. These inserts are tube shaped and made of high-strength steel with two ports for grout injection and a threaded conduit above in order to connect the extension bar. Grouted sleeves are placed inside the precast concrete panel and bars that project from the foundation are positioned inside the sleeves and then fixed in place by grout injection into the sleeves. A typical grouted sleeve connection is shown in Figure 2b.

2.3 **Unbonded post-tensioned connections**

Post-tensioned connections are prestressed using high-strength cables after installation of the walls on the foundation. The cables are usually extended from the bottom of the foundation to the ceiling, joining several precast panel and the foundation together.

3 EXPERIMENTAL PROGRAMME

An experimental programme was developed to assess the seismic behaviour of precast concrete panels connected to the foundation by grouted connections. Different parameters such as wall aspect ratio, SESOC proposed detailing, the type of connections and the magnitude of axial load were included in the experimental programme. As a base case a precast concrete wall was tested with an aspect ratio of three, being an aspect ratio commonly encountered in the review. The metal duct connection type was used in the base case experiment as this is the most commonly encountered type of grouted connections.

3.1 Test specimen details

The specimen had a 3000 mm height and a 1000 mm length, with vertical reinforcement of HD12 spaced at 225 mm c/c, and horizontal reinforcement of HD12 spaced at 250 mm c/c that was uniformly distributed and had 90° hooked ends. Three HD16 bars with a spacing of 400 mm were used to connect the wall to the foundation. The connection bars protruded from the foundation and were embedded inside the wall in 600 mm long corrugated metal ducts that were filled with grout. The other ends of the connection bars were anchored by a 90 degree standard hook inside the foundation. A 20 mm gap between the foundation and the panel was provided by placing two shims underneath the wall. The area around the gap was dry-packed and a day later was filled with grout.

The behaviour of the connection between the wall and the foundation had a dominant influence on the overall force-displacement behaviour of the wall. The moment capacity of the connection was predicted according to the method presented in NZS 3101:2006, with the predicted ultimate lateral force of the connections (F_{cn}) and the ultimate lateral force based on wall flexural response(neglecting the strength reduction factor(F_{wn}) being reported in Table 1. The flexural capacity of the connection and the wall were almost the same, with connection failure expected to occur before any failure in the wall.

3.2 Material properties

Grout and concrete samples were used to determine the compressive strength of the materials. Three grout cubes having dimensions of 50×50×50 mm and three standard concrete cylinders were tested on the same day that the panel was tested. Similar curing conditions were provided by storing the samples next to the wall, with the grout samples placed inside a plastic bag to emulate the condition of the grout inside the metal ducts. The strength of grout (f'_g) and concrete (f'_c)on the day of testing day are presented in Table 1.

3.3 Test setup and instrumentation

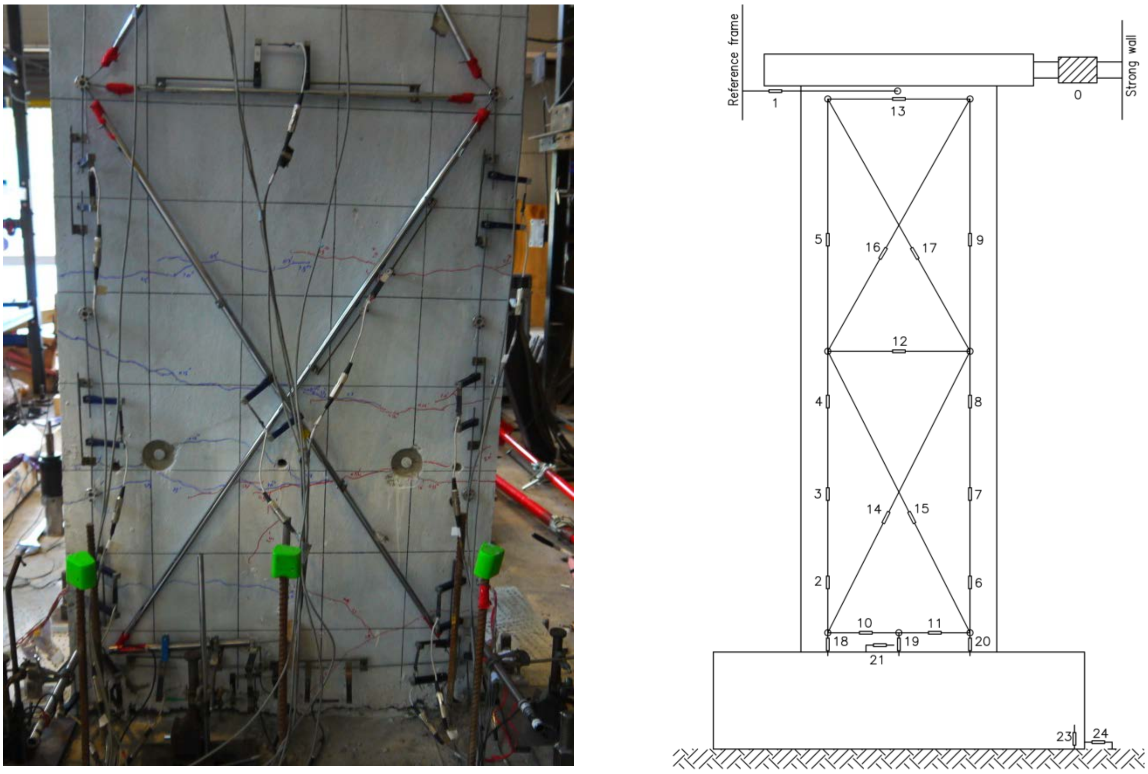
The test setup is shown in Figure 3, which primarily consisted of a reinforced concrete footing and a horizontally mounted hydraulic actuator providing a horizontal shear force to the top of the wall through a 310×310 steel I section loading beam attached to the top of the wall panel. The wall was stabilized from moving in its out-of-plane direction by two struts that were positioned at both ends of the wall with a small gap provided between the loading beam and struts.

Wall instrumentation is also schematically shown in Figure 3. Lateral force was measured by a load cell positioned in series with the hydraulic actuator and the lateral displacement of the top of the wall was measured by a string potentiometer. The shear and flexural deformations of the wall were monitored by displacement gauges labelled in the figure as 3-17. The base uplift and sliding deformations were measured by portal gauges18-20 and 21, respectively. In addition a LVDT (instrument 25) was used to monitor sliding of the panel on the foundation. The out-of-plane movement of both ends of the panel was measured with two other LVDTs (instruments 26, 27). The pull out displacement of two end connection bars from the foundation was monitored by instruments 28, 29. Finally the sliding and uplift of the foundation from the laboratory strong floor were measured by instruments 23, 24.

Embedded strain gauges were used to measure strains of both concrete and reinforcement in the panel and connection, with three reinforcement strain gauges positioned on the bottom, middle and top of the two extreme connection bars. Three other strain gauges were placed on each of the two end vertical reinforcing bars of the panel. Force transfer between connection bars and panel reinforcement was measured by these sets of instruments. In addition, two strain gauges were positioned on transverse bars. Finally, two concrete strain gauges were positioned at each corner of the panel.

3.4 **Testing procedure**

The ACI loading protocol (ACI 2008) was used. The applied cyclic loading started with three cyclic of force-controlled loading and continued with a series of displacement-controlled loading components. Each stage of displacement-controlled loading consisted of three cycles to the selected displacement. Drift value for the next three cycles were to values of between 5/4 times and 3/2 times the previous maximum drift. The drift ratios after force-controlled cycles equalled 0.15%, 0.22%, 0.35%, 0.5%, 0.75%, 1.0%, 1.5%, 2.0% and 2.5%.



(a) The instrumentation (b) The test setup

Figure 3. The test setup and instrumentation.

4 **EXPERIMENTAL RESULTS**

4.1 **General response**

In the first three cycles no crack was formed in the wall and its connection to the foundation. During the fourth cycle, a horizontal crack extending from the in tension corner of the wall toward the centre of the wall appeared. In the next cycles, the cracks propagated beyond the centre of the shear wall and intersected the crack that had formed in the previous cycles. Thus, a single crack crossed the connection. As the test continued, small cracks appeared at the end of the metal ducts (at the height of 600 mm from the connection) at the time the wall was subjected to a one percent drift. These cracks resulted from inappropriate overlap of connection bars with the panel reinforcement. As the loading continued the width of the crack became larger and other small cracks appeared on the wall. The width of the all cracks was less than 1.2 mm and in comparison with rocking of the wall, they had a negligible influence on the overall behaviour of the wall. Crushing the grout layer with yielding and elongation of connection bars allowed the wall to slide on the foundation. The test continued by applying increasing displacement-controlled cycles until the rupturing of a connection bar occurred at 2.5% drift ratio with no concrete crushing occurred during loading. The resultant crack pattern at the end of the experiment is shown in Figure 4a. During the test, no bar pulled out from the metal ducts and no other premature failure was observed. As a result the wall displayed a ductile behaviour with a maximum strength that was approximately 10% larger than the design flexural capacity of the connections.

The contribution of wall sliding on the total drift was low in early stages of the loading but it increased significantly when the wall was subjected to larger drifts. In the last cycles of the test large sliding of the panel was observed that resulted from elongation and failure of the connection reinforcement. More details about sliding are discussed in following sections.

4.2 Force-displacement response

In Figure 4b, the lateral force-displacement of top of panel is plotted. During the three first cycles the wall exhibited an almost linear elastic behaviour. In the fourth cycle inelastic behaviour was observed due to the yielding of the connection bars. After the first yield occurred in a connection bar, the diagram displayed pinching which resulted from opening a gap in the connection zone and allowing the wall to slide. The sliding decreased the stiffness of the wall in the next cycles and finally the experiment finished by rupturing the connection bar that resulted in rapid strength degradation.

The wall exhibited a flexural dominated response. At the maximum force, less than 10 percent of the total drift belongs to sliding of the panel. The results of experiments are summarized in Table 1, where V_{max}, d_{max}, d_{smax} and Δu_{max} are maximum lateral force, corresponding displacement of top of the panel in maximum lateral force, corresponding sliding of the panel in maximum lateral force and corresponding uplift of the panel in maximum lateral force, respectively. At the time of the failure of the connection reinforcement, the residual panel strength was approximately 80% smaller than the maximum panel strength. The ductility of 7 and drift ratio of 2% was achieved by the wall.

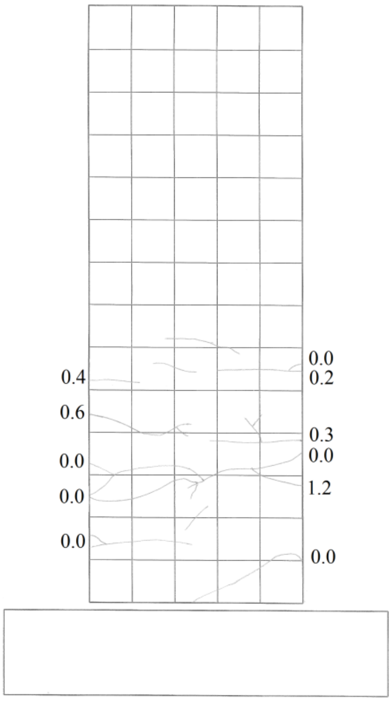
Table 1. Summary of predicted and achieved results

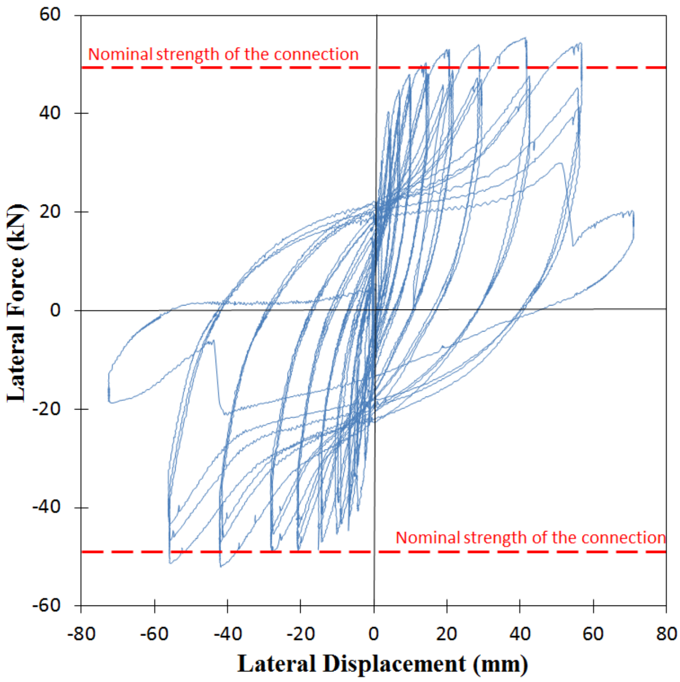
Wall	f'_g (MPa)	f'_c (MPa)	F_{cn} (predicted) (kN)	F_{wn} (predicted) (kN)	V_{max} (measured) (kN)	d_{max} (measured) (mm)	d_{smax} (measured) (mm)	Δu_{max} (measured) (mm)
1	58.1	45.9	48.1	44.3	55.5	41.5	3.1	10.6

4.3 Deformation components

The contributions of four types of wall flexural deformation, wall shear deformation, sliding and rocking on the total force-displacement behaviour of the wall were obtained by displacement gauges that were installed on the wall as shown in Figure 3. The flexural deformation of wall was determined by the curvatures results from the strain of both edges of the wall measured by the displacement gauges. The cross sections of wall assumed to remain plane during loading. The shear deformations of wall were calculated from diagonal displacement gauges according to the approach proposed (Hiraishi 1980) which includes the effect of flexural deformation on diagonal displacement. The sliding of the wall was measured directly by a LVDT and a displacement gauge that were installed at the bottom of the wall. Rocking was computed according to uplift measured by two displacement gauges on both edges of the panel. The total displacement was summed up by these four displacements. The difference between sum of the four types of displacement mechanisms and the measured displacement at the top of the wall is less than 9 percent.

The contribution of each force-deformation mechanisms on the overall behaviour of the wall is shown in Figure 5. Rocking had the largest effect on the behaviour of the shear wall as more than 60% displacement of the top of the panel was due to the rocking. The second and third more important mechanisms are sliding and flexural deformation of the panel. Before the yielding of the connection bars, the flexural deformation of the panel formed approximately 15% of the overall displacement. In larger drifts the yielding and elongation of the connection bars allowed the panel to slide and lessen the flexural deformation. Conversely, the sliding of the panel had more influence on the behaviour of the panel as the panel was subjected to larger displacements. By increment of the sliding length of the wall less tension was transferred through the connection, leading to the reduction in the flexural deformation. The shear deformation of panel had a negligible contribution on the hysteretic behaviour of the panel.





a) Crack pattern and crack width (mm)

(b) Measured force-displacement response

Figure 4. The force-displacement of the panel and the crack pattern and crack width.

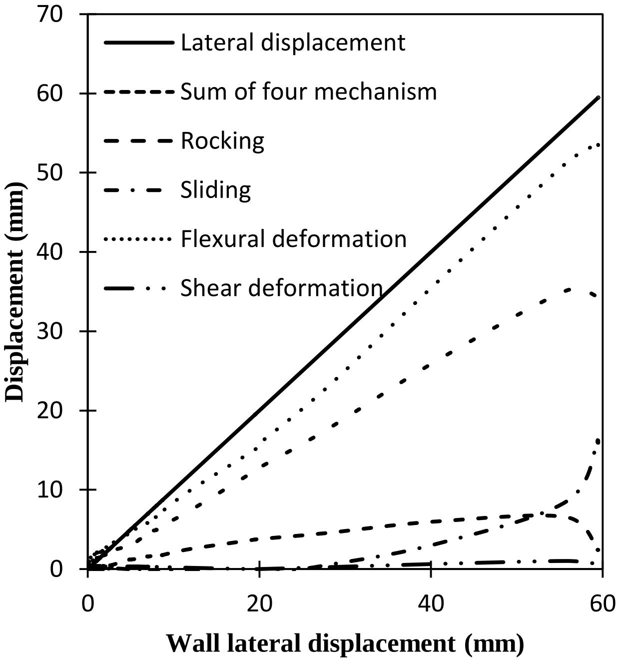


Figure 5. Contribution of each mechanism to the overall displacement.

5 CONCLUSION

A test programme was proposed in order to verify the seismic behaviour of walls connected to foundation with grouted connections. The detailing of the specimens was selected according to the review of more than 4800 precast concrete panel details collected from different precast concrete panel manufactures around New Zealand. As a base case, a precast concrete wall connected to the foundation was tested without using connection confinement that is proposed in SESOC. It observed that the wall displayed a ductile behaviour without occurrences of any premature failures such as bar pull out. The wall experienced low damage and the cracks width did not exceed from 1.2 mm.

It was found that the overall force-deformation behaviour of the precast concrete wall can be considered by summing four deformation mechanisms including rocking, sliding, shear and flexural deformation of panel with acceptable accuracy. Since the wall had a large aspect ratio the behaviour of the wall was dominated by rocking and the influence of the panel deformation was negligible. The sliding of the panel on the foundation would be likely to occur in large drift levels when the connection reinforcement yields which results in degradation of stiffness and pinching of wall force-displacement diagram. Utilizing connectors to increase the resistance of the panel against sliding may improve the behaviour of these connection types.

6 ACKNOWLEDGMENTS

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