In-plane testing of precast concrete wall panels with grouted sleeve

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ABSTRACT: Grouted connections are widely used to connect precast concrete wall panels to their foundations. Various forms of grouted inserts are utilised in different countries to provide a splice between the wall panel and the connection reinforcement. A commonly used coupler insert in New Zealand is grouted sleeve inserts. This type of inserts has a threaded portion on top of the insert to connect the panel reinforcement and a tube-shaped part for positioning and anchoring connection reinforcement using cementitious grout. The two concerns associated with this type of connections are thread slip and reinforcement pull-out from the grout when the inserted reinforcement is subjected to cyclic loads. Thread slip affects the panel stiffness and reduces the connection integrity, and reinforcement pull-out is an undesirable failure mode. In order to evaluate the force-displacement behaviour of connections with grouted sleeve connectors, two full-scale experiments were conducted with one wall panel being reinforced with a single layer of vertical reinforcing and the other wall panel being doubly reinforced. The geometry and detailing of the wall panels was based on a previously conducted review of over 4000 constructed precast concrete wall panels. The specimens were subjected to reverse in-plane cycle loads until failure of either the connection or the wall panel. The results of the experiments are discussed in this article.

1 INTRODUCTION

There has been a wide use of embedded grouted sleeve inserts in precast concrete members of buildings to connect structural elements to each other. The increased speed of construction and reduced material consumption are the main advantages of using embedded connectors in the precast connections. In addition, the reinforcing details can be approximately the same as the conventional cast-in-situ system, and minimal deviation is required from the monolithic concrete design procedure (Haber 2013).

A commonly used insert in New Zealand precast concrete buildings is called grouted sleeve insert which have been used in more than 35% of grouted connections between precast concrete wall panels and foundations (Seifi et al. 2015). Grouted sleeve insert has a threaded portion on top of the insert to connect the wall panel reinforcement and a tube shape part for positioning and anchoring connection reinforcement using cementitious grout. A concern associated with the use of this type of connectors is thread slip at the top of the connectors that decreases the stiffness of the wall panels with grouted sleeve connections when they are subjected to in-plane lateral loads. Another concern regarding on this connectors is reinforcement pull-out when it is subjected to cyclic loads.

An objective of this research is to evaluate the influence of grouted sleeve connectors on the seismic performance of the precast concrete wall panels. The overall load-displacement performance of the precast concrete wall panels is a sum of wall panel deformation (shear and flexural deformation) and connection displacement (rocking, sliding) (Becker et al. 1980). The detailing of the connection can alter the contribution of the connection displacement in the system leading to different seismic design parameter such as energy dissipation, and failure drift (Seifi et al. 2016). Hence, full-scale experiments were conducted to investigate the behaviour of grouted sleeve connection in most commonly used detailing in New Zealand. The detailing of wall panels were according to the previously conducted review of constructed detailing in New Zealand described in Chapter 2. It was found that the grouted sleeve connection in precast concrete wall panels were generally in two types (Seifi et al. 2016). In first type which is more commonly used in double layer reinforced wall panels the connection reinforcement, the bars connected to the grouted sleeve inserts are extend as much as its development length, and separate reinforcement are used to reinforce the wall panel. In this type of detailing the reinforcement content of connection is usually less than wall panel reinforcement, forming “joint structure”. In the second method the bars connected to the inserts are extended to the top of the wall panel, and function as a vertical reinforcement of the wall panel. Therefore, in this detailing the reinforcement of the connection is usually same as the wall panel reinforcement, and the seismic behaviour is similar to monolithic wall panels.

In the present study, in order to verify the behaviour of the grouted sleeve inserts two series of experiments were conducted. In the first series of the experiment programme, two full-scale wall panels connected to the foundations with grouted sleeve connections were tested with applying a lateral cyclic load in order to evaluate the influence of grouted sleeve connectors on the wall panel performance. In the second series of the experiments, non-embedded grouted sleeve inserts were tested with applying a monotonic tensile axial force to them. The results of the conducted experiments are discussed.

2 EXPERIMENTAL PROGRAMME

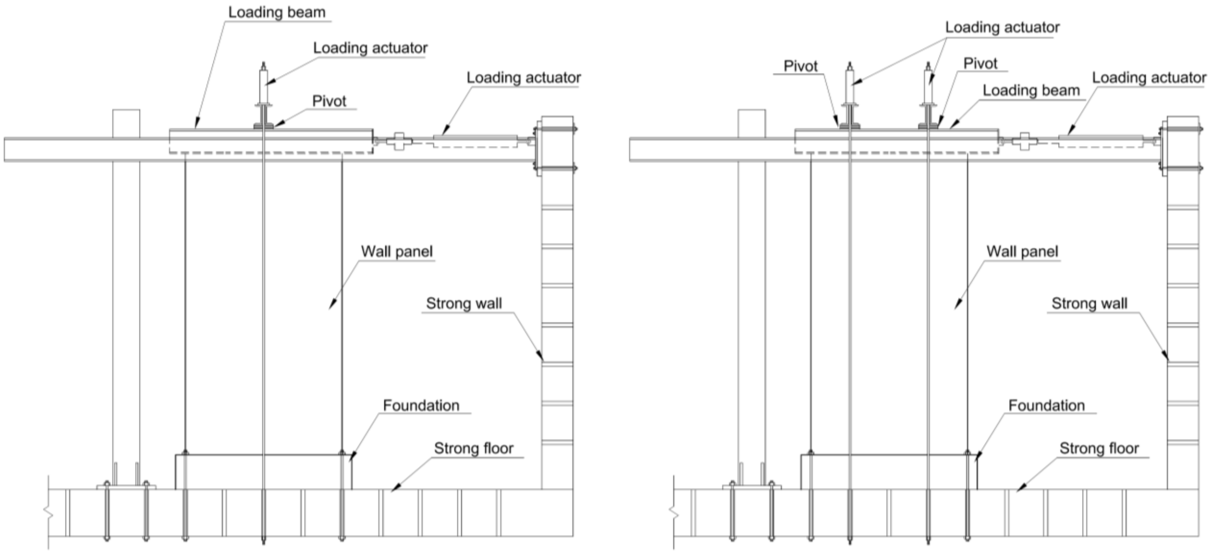
In order to examine the seismic behaviour of grouted sleeve connections, an experimental programme has been designed and conducted in two phases. In the first phase, two full-scale experiments were conducted with applying both axial and lateral loads to the wall panels. In the second phase, coupler tests were performed with applying a monotonic tensile load to couplers.

2.1 Full-scale experiments

Two full-scale precast concrete wall panels were tested to examine the in-plane seismic behaviour of precast concrete wall panels connected to a foundation by the grouted sleeve connection. The projected starter bars from the foundation were placed inside the grouted sleeve inserts which were positioned inside the wall panel during the construction of the wall panel. The other end of the connection reinforcement was anchored by 90-degree standard hook inside the foundation. The gap between wall panel and the foundation were sealed with dry-pack and a day later was filled with pumping non-shrinkage grout into the inserts. Both wall panels had a four metre height and a two metre length with a thickness of 150 mm for Panel 1 and 200 mm for Panel 2. Panel 1 was reinforced horizontally with single layer of grade 500 HD12 spaced at 250 mm, and the vertical reinforcing bars had a 16 mm diameter and a spacing of 300 mm that were connected to the top of grouted sleeve inserts. Panel 2 was reinforced with double layer grade 500 HD12 and a spacing of 240 mm. In both wall panels, the connection between the wall panel and the foundation was composed of HD16 rebar with a spacing of 300 mm.

2.1.1 *Test Setup*

The test setup primarily consisting of wall panel, foundation, steel I section beam connected to the top of the wall panel, perpendicular beams, and tendons for applying axial load, and a horizontal mounted hydraulic actuator providing the horizontal cyclic lateral load. The details of the test setup are schematically shown in Figure 1. The axial load was applied to the wall panels through post-tensioned tendons that were installed in both sides of the wall panels. Two tendons were used for Panel 1 and four tendons were used for Panel 2 as a larger axial load was required. The force of the tendons was monitored and adjusted during each experiment to keep the axial force in ±5% tolerance. One end of tendons were connected to a beam that were positioned perpendicularly on top of the steel I section beam, and the other end of tendons was connected to the strong floor. A hydraulic actuator was connected to each tendon in order to apply axial force to them. In order to minimize application of any moments to the top of wall, a pivot was used under the perpendicular beams. Two channel beams were used to prevent the out-of-plane movement of wall panels. One end of the channels was connected to a column positioned beside the wall panel and the other end was connected to the strong wall.



(a) Panel 1 (b) Panel 2

Figure 1. The details of the full scale test setup

2.1.2 *Instrumentation*

The walls were instrumented to monitor important aspects of wall panel response when subjected to in-plane lateral loads, with the lateral load measured by a load cell placed in series with the actuator. Additional load cells were installed between the post-tensioned bars and the strong floor to measure the applied axial load, and the lateral displacement at the top of the wall panels was measured by a string potentiometer. In-plane rocking deformations of the wall panels were measured using three displacement gauges that were positioned at the two ends and at the middle of the connection between the wall panel and the foundation. In addition, the relative in-plane sliding displacement between the wall panel and foundation was monitored by a displacement gauge and a LVDT installed midway along the connection. Shear and flexural deformations of the wall panels were measured by 16 displacement gauges installed on each wall panel, and two displacement gauges were used to measure sliding and uplift of the foundation relative to the laboratory strong floor.

Embedded strain gauges were utilized in each wall panel to measure reinforcement strains at critical locations, with the pattern of embedded strain gauges. Three reinforcement strain gauges were positioned at the bottom, middle and top of the two outside connection bars extending from the foundation, at elevations of 20 mm, 200 mm, and 400 mm above the connection level. In addition, three reinforcement strain gauges were placed on the two outermost vertical bars of each wall panel. In the wall panels where confinement reinforcement was provided to the connection, two additional reinforcement strain gauges were attached to the bottom stirrups that confined the two extreme connection bars. In addition, two concrete strain gauges were positioned at heights of 150 mm and 350 mm above the base of each wall.

2.1.3 *Material properties*

Concrete and steel reinforcement samples were taken during construction of the wall panels and grout samples were collected during grouting of the connections. The reinforcement samples were tested by applying monotonic axial tensile loads to the samples. Three grout cube samples with dimensions of 50×50×50 mm were tested for each wall panel. In addition, three concrete compression tests were performed on cylinder samples with a radius of 100 mm and a height of 200 mm. Concrete samples were subjected to similar curing conditions as for the wall panels by placing them next to each wall panel. The grout samples were kept inside a plastic bag to emulate the condition of the utilised grout inside the metal ducts. Grout and concrete samples were tested on the same day as the wall panel was tested. The measured material strengths are summarised in Table 1.

Table 1. Properties of utilized materials (all stresses in MPa units)

Wall panel number	Grout strength	Concrete strength	Connection reinforcement		Panel reinforcement	
			Yield stress	Ultimate stress	Yield stress	Ultimate stress
1	56	39	464	627	516	654
2	55	40	464	627	516	654

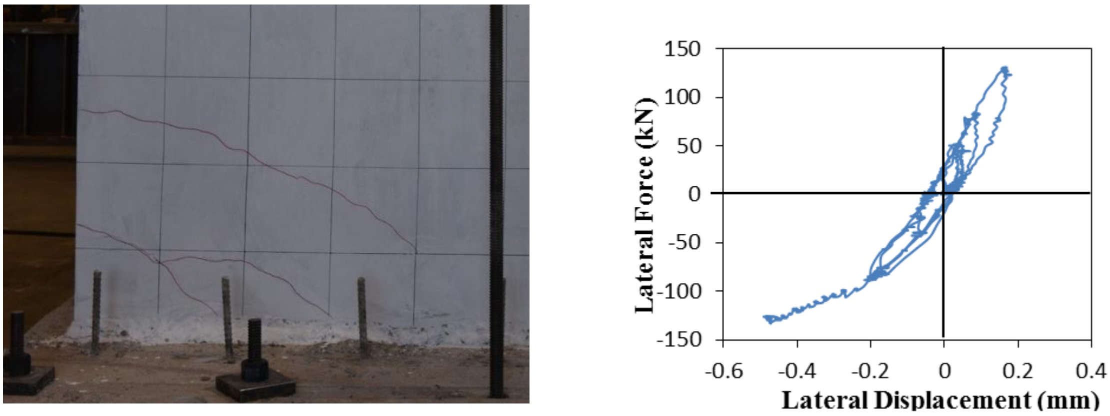
2.1.4 *Testing procedure*

The ACI loading recommendations (ACI 2008) was used to determine the loading sequences of the experiments. The loading started with three force-controlled loading cycles and continued with a series of displacement-controlled loading cycles until failure. The failure point was defined as the point where the stiffness had decreased to less than 10% of the initial stiffness or the lateral force had decreased to 80% of the maximum lateral force. Three cycles to the selected drift value were applied at each stage of the displacement-controlled loading with the selected displacement-controlled drift values being 0.2%, 0.25%, 0.35%, 0.5%, 0.75%, 1.0%, 1.5%, 2.0%, and 3.0%.

2.1.5 *General response*

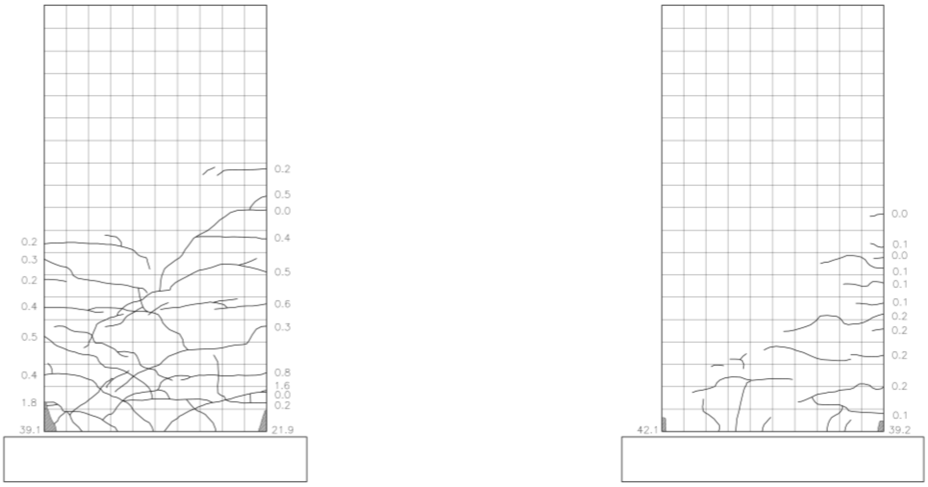
No crack was formed on Panel 1 during applying first two force-controlled cycles, and at the third cycle two cracks were appeared with the crack width of was 1.0 mm and 0.3 mm at the height of 200 and 500 mm from the connection, and caused a considerable stiffness reduction of the wall panel. The crack resulted from the thread slip between the connected reinforcement and the grouted sleeve insert. The image of the formed cracks and force-displacement diagram in this stage of the loading are shown in Figure 2. In the next cycle, the thread slip occurred at the extreme grouted sleeve insert located at the other vertical edge of the wall panel causing formation of a symmetric crack pattern. The width of the cracks in two vertical edges of the wall panels was different as the extent of the thread slip was different in two extreme grouted sleeve inserts. When the 0.5% drift level was applied to the wall panel new cracks appeared on one vertical edge of the wall panel but in the other edge only the width of cracks became larger. This behaviour can be attributed to the larger extent of thread slip at the one of the extreme grouted sleeve insert of the wall panel to the other, which resulted in a smaller proportion of wall panel deformation and the formation of fewer cracks. At the drift level of 0.75%, new cracks were appeared in both vertical edges of the wall panel, and the maximum width of cracks reached to 1.4 mm. In the next drift level of 1.0% new cracks were formed on the vertical edge that had fewer cracks, hence the crack pattern was quite symmetric on the wall panel. This behaviour was due to decreased contribution of thread slippage when a larger drift level was applied to the wall panel. Cracks grew in the 1.5% drift level and the compression toe of the wall panel initiated to spall. The largest crack width was 1.8 mm in this drift level. During application of the 2.0% drift level, reinforcement pull-out occurred in the both extreme grouted sleeve inserts causing failure of the connection and decrement in the wall panel cracks width. The experiment continued with application of the 3.0% drift level that caused reinforcement pull-out from all other inserts of the connection.

Rocking dominated the overall response of Panel 2, which was reinforced with a double layer of longitudinal bars. No crack was formed on the wall panel when the applied drift levels were below 0.5%. This behaviour was attributed to the increased vertical reinforcement content in Panel 2 which significantly increased the relative stiffness of the wall panel in comparison to the connection stiffness. At 0.5% drift level, two cracks with a width of 0.1 mm were appeared at the vertical edge of the wall panel at the elevation of 700 mm and 1000 mm from the connection level. In the meantime, no cracks formed on another vertical edge of the wall panel. At the next drift level of 0.75%, several more cracks were appeared with the maximum width of 0.4 mm on the walls, and concrete at the compression toe of the wall panel spalled. At the drift level of 1.0% no new crack was observed on the wall panel. At the next drift level of 1.5%, the more extensive concrete spalling was observed and reinforcement pull-out from grouted sleeve inserts were occurred which resulted in closure of many cracks. At the next drift level of 2.0%, no new crack and concrete spalling occurred. The experiment conducted at 3.0% drift level when two connection bars were ruptured and other five connection bars were pulled out from grouted sleeve inserts. The crack pattern at the end of the experiment is shown in Figure 3.



(a) Cracks resulted from threaded insert (b) Stiffness reduction caused by thread slip

Figure 2. Thread slip at early stages of loading



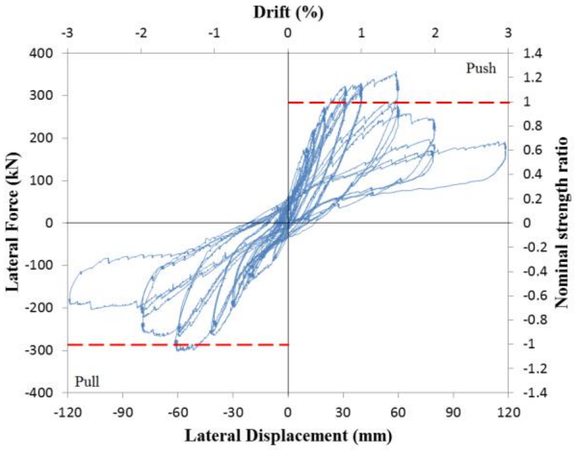
(a) Panel 1 (b) Panel 2

Figure 3. Crack patterns for two tested wall panels, and the maximum crack width

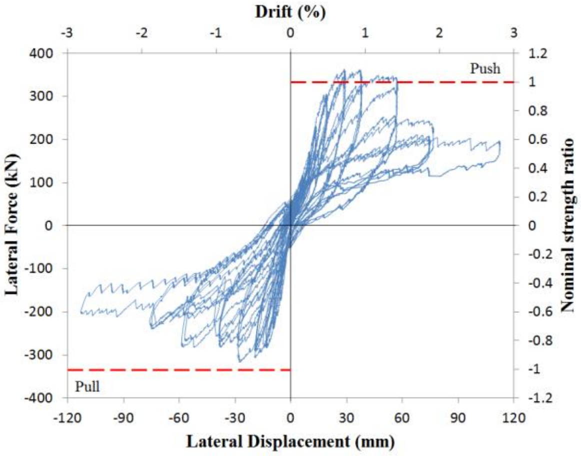
In order to evaluate the quality of the grouting inside the inserts, two grouted sleeve inserts were taken off from the wall panel and they were cut vertically. Reinforcement pull-out occurred in one of the inserts and in the other insert reinforcement rupturing occurred. A large amount of spalled grout came out from the grouted sleeve insert that pull-out occurred that was due to cyclic movement of bar after the pull-out occurred. In contrast, limited grout damage was observed in the grouted sleeve insert that bar fracture occurred. It can be concluded that the grouted sleeve inserts were properly filled with grouted.

2.1.6 *Force displacement behaviour*

The lateral force-displacement diagrams of two tested wall panels are shown in Figure 4. During first two cycles, an almost linear elastic behaviour was observed in the Panel 1. In the third cycle, a large displacement was recorded in one side of the load-displacement diagram of wall panel. The reason for this behaviour was the larger thread slip that occurred in one side of the wall panel, and it decreased the stiffness of the wall panel in the one side of the force-displacement diagram. Connection reinforcement yielding occurred in the fourth cycle and it caused an inelastic behaviour and pinching of the diagram. The diagram pinching became larger when larger drifts were applied as connection reinforcement yielding and elongation opened a gap in the connection zone. At the drift level of 1.5%, the lateral force reached to the maximum magnitude of 350 kN in one side of the load-displacement diagram and 302 kN in the other side of the diagram. The maximum lateral force was greater than the connection nominal strength (283kN) in both sides of the load-displacement diagram. In the next drift levels, reinforcement pull-out caused larger pinching in the diagram. As loading continued, more reinforcement pulled out from the inserts and it decreased the stiffness of connection. Consequently, at the final cycle of 3.0% drift level the lateral force was 187 kN which was 52% of the maximum lateral force.



(a) Panel 1



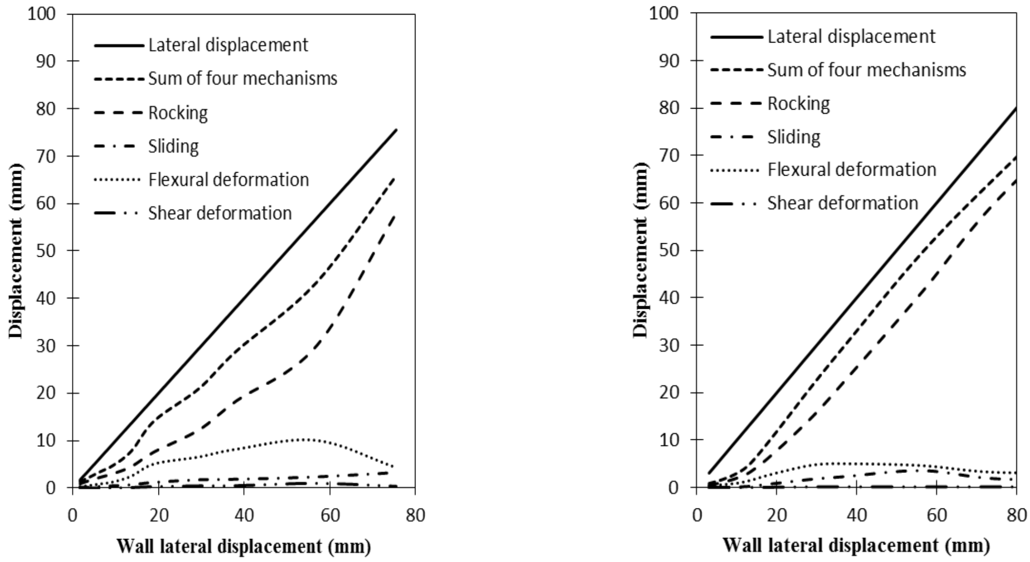
(b) Panel 2

Figure 4. Hysteresis response of for wall panels

Larger lateral forces were measured in Panel 2 in comparison with the previous experiment due to the greater applied axial load. During first three cycles, an almost linear behaviour was observed but the stiffness of two sides of the load-displacement diagram was different due to the difference in thread slip at the two extreme grouted sleeve inserts. Hence, the wall panel stiffness was about 61% more in one side of the force-displacement diagram than the other side. As loading continued, yielding of connection reinforcement occurred and it caused nonlinear load-displacement diagram. At the drift level of 0.75%, the wall panel reached to the maximum lateral force of the 354 kN and 316 kN in each direction which was below the connection nominal strength of 335 kN in one side of the force-displacement diagram. At the drift level of 1.0%, reinforcement pull-out occurred in both extreme connection reinforcement causing a reduction in the lateral force of the wall panel. In the next drift level of 1.5%, the force-displacement diagram displayed more degradation and the lateral force, and the lateral force decreased to the magnitude of 343 kN and 263 kN in each direction. Finally, at the drift level of 2.0% two extreme connection bars fractured, and caused a significant reduction in the lateral force of the wall panel. In this stage of loading, the lateral force was 237 kN and 239 kN which were about 67% of the maximum lateral force. Although failure occurred at the drift level of 2.0%, the experiment continued with application of 3.0% drift. All remaining connection reinforcement pulled out from their inserts in this drift level. A steeper unloading curve and consequently a smaller area and energy dissipation were obtained in Panel 2 was attributed to the larger axial load in Panel 2 caused

2.1.7 *Deformation component*

Four force-displacement mechanisms of rocking, sliding, shear and flexural deformation of the wall panel are expected, when in-plane lateral loads were applied to the precast concrete wall panel connected to the foundation (Becker et al. 1980). The contribution of each mechanism on the wall panel behaviour was calculated by data obtained from displacement gauges. The flexural deformation of the wall panels was determined by measuring the wall panel curvature caused by the strain difference in two edges of the wall panel. It was assumed that the cross sections of wall panels remain plane during loading, according to the method proposed by Hiraishi (1984). The shear deformations of wall panels were obtained from diagonal displacement gauges according to the same method (Hiraishi 1984), which includes the influence of flexural deformations on the diagonal displacement. The sliding of the wall panels was determined directly by a LVDT and a displacement gauge that were installed at the bottom of the wall panels. Rocking was calculated using the measured uplift with two displacement gauges on two extreme ends of the connection. In Figure 5, contribution of each type of displacement mechanism is shown. The sum of four displacement mechanisms was compared with the displacement measured at the top of the wall panel. The difference between sum of the four displacement mechanisms and the measured displacement at the top of the wall panels was less than 13%.



(a) Panel 1 (b) Panel 2

Figure 5. Contribution of each load displacement mechanisms

The behaviour of Panel 1 was mostly governed by rocking and flexural deformation of wall panel that had 52% and 23% contribution in the total behaviour of the wall panel, respectively. The contribution of wall panel flexural deformation was larger when the lateral displacement was below 60 mm (1.5% drift) but when a larger displacement was applied, the flexural deformation of the wall panel was decreased. This behaviour was due to reinforcement pull-out which decreased the stiffness of the connection and consequently less moment was transferred to the wall panel. At the meantime, the lower connection stiffness increased the wall panel rocking. Sliding and shear deformation of the wall panel had a negligible effect on the wall panel performance.

Rocking had the largest contribution in both wall panels behaviour as 80% of the lateral wall panel displacement was due to rocking. The larger contribution of rocking in Panel 2 was due to the larger thickness of the Panel 2 that decreased the contribution of wall panel flexural deformation. The wall panel flexural deformation had approximately 12% contribution in the total deformation of the wall panel. Similar to Panel 1 there was a decrease in the magnitude of wall panel flexural deformation when the connection bars pulled out from the grouted sleeve inserts. Sliding and shear deformation of the wall panel had negligible impact on the wall panel performance.

2.2 **Tensile test of grouted sleeve insert**

Two grouted sleeve connectors were tested with applying a monotonic tensile force in order to evaluate the performance of the grouted sleeve inserts. Two Grade 500 reinforcing bars with diameter of 16 mm were spliced using a grouted sleeve insert. The insert was filled with high-strength grout, and three cubic grout samples with a dimension of 50×50×50 mm were provided. The average of test-day compressive strength of the three grout samples was 71 MPa. A monotonic tensile force was applied until failure, which was due to reinforcement rupture in both experiments. The average yield and ultimate strength for the two experiments was 517 MPa and 627 MPa, respectively. No thread damage was observed on the grouted sleeve insert and the attached reinforcing bar, which were well designed until the failure force.

3 CONCLUSIONS

The response of precast concrete wall panels with grouted sleeve connections was experimentally examined. The following conclusions were drawn:

- The thread slip was observed when the grouted sleeve connections were subjected to cyclic forces that resulted in significant reduction of the wall panel stiffness. A method should be used to limit the extent of thread slip.
- The behaviour of both wall panels was dominated by rocking and the wall panel flexural deformation and sliding had less impact. The sliding of the wall panels on the foundation would be likely to occur in large drift levels when the connection reinforcement yields and a gap opened in the connection zone.
- It was found that the overall load-deformation behaviour of the wall panel could be considered by summing four deformation mechanisms including rocking, sliding, shear and flexural deformation of wall panel with acceptable accuracy.
- The coupler behaved adequately when they were subjected to the monotonic tensile loads but coupler failure was observed when cyclic forces were applied. It was concluded that the behaviour of the grouted sleeve couplers depended on the loading history.

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