

In-plane seismic testing of precast concrete wall panels with grouted metal duct base connections

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Abstract

In order to achieve satisfactory seismic performance, the connections between precast concrete wall panels and other structural elements should be well-designed to avoid brittle connection failure during an earthquake. Following the 2010/2011 Canterbury (New Zealand) earthquakes the seismic performance of grouted connections used for precast concrete wall panels was questioned. The brittle connection failure during the earthquake resulted in recommendations for more robust detailing of grouted metal duct connections. A set of experimental tests was performed to investigate the seismic behaviour of both existing and newly recommended detailing of precast concrete wall panels. Testing was comprised of seven full-scale precast concrete wall panels with wall-to-foundation grouted metal duct connections that were subjected to reversed cyclic in-plane lateral loading. Walls with existing connection detailing were found to perform adequately when carrying low axial loads, but performance was found to reduce as the axial load and wall panel length increased. The use of transverse confinement reinforcement around the grouted metal ducts was observed to prevent brittle connection response and to improve the robustness of the reinforcement splice.

Author keywords: Precast wall panels; Seismic design; Metal duct pull-out; Confinement; Energy dissipation.

INTRODUCTION

Precast concrete members are widely used in many countries for structural forms ranging from low-rise warehouses to high-rise multi-storey buildings. In this form of construction, the structural concrete components are cast off-site and are then assembled at the construction site. The advantages of utilizing precast concrete elements are cost savings, better quality control, increased speed of construction, reduced material consumption, and the potential to use high strength concrete (PCI 2010). Precast concrete walls are commonly used as a primary force resisting system in a number of countries, including US, Japan and New Zealand (Hawkins and Englekirk 1987), because of their significant stiffness and strength against lateral forces deriving from earthquake and wind loads. The structural behaviour of precast concrete walls is a combination of wall behaviour and connection behaviour (rocking and base sliding) (Becker et al. 1980), with the contribution of connection behaviour on the global seismic performance of the wall being dependent on the detailing of the connection and its relative strength and stiffness.

One method that is used in New Zealand to connect precast concrete wall panels to their foundations entails the use of metal duct grouted connectors, with an example of a metal duct connection shown in Fig. 1a (Seifi et al. 2016). In this type of connection the starter bars from the foundation are positioned inside metal ducts with a thickness of 0.3 mm that are cast inside the wall panel, and then the metal ducts are filled with grout at the construction site, as shown in Fig. 1b. The main advantage of the metal duct grouted connection is the simplicity of this detail, but the vulnerability of the connection detail was revealed during the 2010/2011 Canterbury earthquakes (SESOC 2013). Fig. 2 shows an example of metal duct connection damage during the 2010/2011 Canterbury earthquakes. To increase the seismic robustness of the grouted connection between precast wall panels and the foundation, the Structural Engineering Society of New Zealand (SESOC) has recommended new detailing for this connection type (SESOC 2013). The proposed detailing requires that rectangular stirrups be used around ducts to provide confinement for the connection and also to improve the robustness of the splice between the connection and the vertical wall reinforcement, as shown in Fig. 3. It is generally accepted that confinement improves the strength of the splices (Riva 2006).

In the present study seven precast concrete wall panels with conventional reinforcement detailing were tested to verify the seismic performance of the panel-to-foundation connection when using grouted metal ducts. The seismic performance of the wall panels is discussed, including details of load-displacement behaviour, crack patterns, and wall-panel failure modes.

BACKGROUND AND PREVIOUS RESEARCH

The vulnerability of metal duct connections when used in precast concrete walls has been demonstrated in several past studies (Riva et al. 2006; Kim 2000) and also during the 2010/2011 Canterbury earthquakes in New Zealand (SESOC 2013). As an example, Crisafulli et al. (2002) tested a reinforced concrete shear wall that was connected to its foundation using metal ducts, in order to evaluate the seismic performance of walls having a lightly reinforced metal duct connection. The results of the experiment indicated that the wall lateral stiffness decreased significantly at small lateral drifts, and large residual displacements were measured during the experiment. In other research, the reversed cyclic behaviour of columns connected to their foundation using grout filled metal ducts was compared with the behaviour of cast-in-situ monolithic concrete columns (Kim 2000). It was found that the grouted metal duct connection performed poorly due to deterioration of the longitudinal bar splice, which caused reinforcement pull-out from the metal duct. Riva (2006) tested grouted-sleeve and grouted-pocket column-to-foundation connections, and compared the behaviour of two precast concrete columns having different connection types with the behaviour of cast-in-situ reinforced concrete columns. Cyclic horizontal displacement was applied to the top of the columns to generate cyclic moments at the column-to-foundation connection, and a constant axial load was applied. Although all columns had almost the same flexural capacity, their energy dissipation and failure displacement were different depending on the level of confinement provided to the connections, with enhanced ductility and lower pinching observed in the connections that had a larger confinement level. It was established that metal duct connections generally performed poorly as the column stiffness decreased due to the formation of large cracks that spread around the metal ducts.

Contrary to the above studies where unfavourable behaviour was identified, several studies have been undertaken where the use of metal duct connections has led to favourable behaviour being exhibited. A comprehensive literature review on the behaviour of different types of connections used in bridge bent caps was conducted by Restrepo et. al. (2011), where it was found that grouted metal duct connections used in the bent cap had a linear behaviour that resulted in an extensive drift being achieved, with plastic hinging forming in the column. Restrepo et. al. (2011) recommended that transverse joint shear reinforcement be used to achieve full ductile behaviour of grouted metal duct connections.

The behaviour of an innovative grouted socket connection between cap beams and unbonded pre-tensioned columns subjecting to lateral seismic forces was examined by Thonstad et al (2016). It was found that most damage occurred at the top and bottom of the column, and that the connection behaved well with minimal damage experienced.

PRECAST CONCRETE WALL SURVEY

A review of detailing used in recently manufactured precast concrete wall panels was conducted by collecting data from precast concrete manufacturers in three major New Zealand cities of Auckland, Wellington and Christchurch. This review involved categorising more than 4800 wall panels used in 108 projects based upon geometry and reinforcement content of the wall panels, and the specific characteristics of connections. 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 the connection. 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 (Seifi et al. 2016).

The most commonly observed wall panel configuration, representing 42% of all walls, had a 150 mm thickness and was reinforced with a single layer of vertical and horizontal bars. Double-layer reinforced wall panels with a 200 mm thickness were documented in 25% of the reviewed detailing. The remaining 33% of wall panels had a thickness equal to or larger than 225 mm. The wall panel height to length aspect ratio was observed to most commonly range between 3 and 5, and wall panels were typically subjected to an axial load ranging from 0% to 10% $A_g f'_c$. Grade 500 MPa HD12 bars spaced at 250 mm

both horizontally and vertically was a common reinforcement detail. For panel-to-foundation metal duct connections, the most commonly used reinforcing bar area was found to be between 0.4% and 0.6% of the gross wall panel cross section, and the most commonly-used reinforcement was grade 500 MPa HD16 with a spacing of 400 mm to 450 mm. The wall panels should be designed to have an inter-storey drift of below 2.5% when subjected to the ultimate limit state (ULS) seismic force, based on NZS 1170.5 (2004). However, it should also be noted that drift demands may exceed this limit during a maximum considered earthquake, and that curvature limits for plastic hinge regions of different ductility class in the New Zealand Concrete Structures Standard, NZS 3101:2006, will often govern the design.

EXPERIMENTAL PROGRAMME

An experimental programme was developed to examine the seismic performance of precast concrete wall panels with wall-to-foundation grouted metal duct connections. Different parameters such as reinforcement details, wall panel thickness and aspect ratio, magnitude of axial load, and use of the proposed confining stirrups were included in the experimental programme.

Test specimen details

The reinforcement and dimensions of the test wall panels were selected according to the most commonly encountered details identified from the review exercise. All of the tested wall panels had a height to length aspect ratio of between 2 and 3, which resulted in the dominant seismic response being associated with rocking and flexural behaviour. The geometry and reinforcement details of the tested wall panels and the applied axial load are summarised in Table 1. The first four wall panels represented perimeter walls in industrial warehouse buildings, with a zero applied axial load and a height to length aspect ratio of 3. These four wall panels represented panels from the perimeter wall of warehouse buildings where the weight of the light steel roof is negligible in comparison to the selfweight of the precast concrete wall panels. In Wall 1 no confining reinforcement was provided in the region of the panel-to-foundation grouted metal duct connection, whereas Wall 2 and Wall 3 had the same geometry as for Wall 1 but incorporated two different shapes of stirrups within the connection region. All wall panels except Wall

4 had a 150 mm thickness and vertical reinforcement as a single layer of HD12 (lower 5% characteristic yield strength of 500 MPa) spaced at 225 mm, whilst Wall 4 had a 200 mm thickness and was reinforced with a double layer of HD12 spaced at 225 mm. Wall 4 was reinforced with a double layer of reinforcing bars to replicate the detailing commonly used in New Zealand. Wall 5 had the same geometry and reinforcement detailing as for Wall 1 but was tested with the application of a moderate axial load of $0.05A_g f'_c$. Similarly, Wall 6 and Wall 7 were intended to represent wall panels in the lower levels of multi-storey buildings, which usually have a height to length aspect ratio of less than 3 and a moderate level of axial load. These wall panels had a height to length aspect ratio of 2 and were tested with the same level of axial load as was applied to Wall 5. The connection in Wall 7 was confined with the placement of rectangular stirrups around the metal ducts, and for all tested wall panels the vertical and horizontal reinforcement was anchored at the edges of the wall panel with a 90° standard hook.

To connect the wall panel to the foundation, starter bars from the foundation were embedded inside 600 mm long metal ducts that were later filled with non-shrinkage grout. The other end of the connection reinforcement was anchored inside the foundation using a 90 degree standard hook. The wall panel was initially erected on top of the foundation by providing a 20 mm gap underneath the panel. The area around the gap was dry-packed and a day later was filled by pumping non-shrinkage grout into the metal ducts.

Test setup

Two different test setups were used for testing the wall panels. For test specimens without applied axial load, the test setup primarily consisted of a reinforced concrete footing, a precast concrete wall panel, and a horizontally mounted hydraulic actuator providing the horizontal cyclic lateral force. The movements of wall panels were restrained in their out-of-plane direction by two parallel H shape steel columns that were positioned on each side of the wall panel. The details of the test setup are shown in Fig. 4a. Both sides of the gap between the H shape steel columns and the loading beam were lubricated with oil to minimize friction forces.

A different test setup was used in the experiments where axial load was applied, as the application of axial load was achieved using two post-tensioned bars that were placed on each side of the wall panel. During each of these latter experiments the bar force was adjusted to keep the applied axial force to within $\pm 5\%$ of the target force. The post-tensioned bars were connected to a beam that was placed perpendicularly on top of the steel I section beam positioned on top of the wall panel, and a pivot was used between two beams in order to prevent application of any out-of-plane moments to the wall panel. Two channel section beams were installed on each side of the wall panel in order to prevent out-of-plane wall panel movement. One end of the beams was connected to the strong wall and the other end was connected to a column placed at the other end of the wall panel. The beams restrained movement of wall panels in their out-of-plane direction. The details of the second test setup are shown in Fig. 4b.

Instrumentation

The layout of instrumentation is shown in Fig.5. 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. Two 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. To measure the lateral drift of post-tensioned bars during testing of Panel 5 another string potentiometer was used to monitor the lateral displacement that occurred at the top of the post-tensioned bars due to movement of the wall panel. Because the magnitude of this displacement was found to be negligible, a decision was made to not measure this displacement during the final two experiments. 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 shown in Fig. 5. 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, as shown in Fig. 5. In addition, two concrete strain gauges were positioned at heights of 150 mm and 350 mm above the base of each wall.

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 2.

Testing procedure

A loading protocol based on the ACI ITG-5.1 recommendations (ACI 2008) was used to determine the applied loading sequence. The loading started with three force-controlled loading cycles and continued with a series of displacement-controlled loading cycles until failure. The force at the first three force-controlled cycles was below 0.6 of the nominal connection strength according to ACI ITG-5.1 recommendations (ACI 2008). 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 0.15%, 0.2%, 0.25%, 0.35%, 0.5%, 0.75%, 1.0%, 1.5%, 2.0%, 2.5%, and 3.5%. Because the loading was quasi-static the impact forces resulted from wall panel movement were not considered in this experiment programme.

EXPERIMENTAL RESULTS

General response

The behaviour of the wall panels varied dependent on the wall panel height to length aspect ratio and the magnitude of axial load applied to each wall panel. The crack patterns of the seven tested wall panels are shown in Fig. 6 and Fig. 7 and a summary of observed behaviour during the experiments is also reported in Table 3. A summary of observed response of Wall 1-4 are presented below:

- Walls 1-3 behaved similarly, with their behaviour dominated by wall panel rocking on the foundation, and noticeable sliding was also observed at the wall panel base. The different detailing of connection confinement reinforcement did not affect the overall behaviour of walls 1-3. In each case the connection reinforcement fractured during cycles to 2.5% drift at which point testing was concluded. No significant concrete spalling or crushing was observed in each of the three tested wall panels.
- At a drift level of 2% different crack patterns were observed across the three wall panels, with this variation being attributed to differences in the extent of out-of-plane displacement that developed at the base of each panel. For each test this out-of-plane displacement was attributed to a combination of bond slip and plastic deformation of the connection reinforcement as a gap opened between the wall panel and the foundation during loading of the connection in tension. During reversed loading this gap facilitated out-of-plane deformation at the base of the wall panel when the connection was loaded in compression, with the different extents of out-of-plane displacements resulting in different crack patterns on the three wall panels.

- the overall response of Wall 4 was dominated by rocking and wall panel sliding, and the panel remaining undamaged during testing of Wall 4. The experiment concluded at a drift level of between 2.0% and 2.5% when two of the outer connection bars fractured.

The effect of axial load

The application of a moderate level of $0.05A_g f'_c$ axial load to Wall 5-7 changed the performance of wall panels in terms of extend of wall panel sliding, concrete spalling, and the failure type. The summary of observed response of Wall 5 is presented below:

- The overall response of Wall 5 was dominated by rocking, but fewer cracks appeared in the Wall 5 panel when compared to Wall 1 as the applied axial load prevented the formation of cracking.
- In comparison with the previous four experiments, less wall panel sliding was measured. This was attributed to axial load being applied to the wall panel, which contributed to closing of the gap between the wall panel and foundation and resulted in increased friction being developed along the connection between the wall panel and the foundation.

The effect of wall panel length

The length of wall panels affected the failure mechanism of the wall panels with grouted metal duct connections. The increase of wall panel length increases the length of compression toe of the and may resulted in metal duct pull out from the wall panel. As a result, different performance was observed for Wall 6, which had a larger length than for the previous five wall panels (see Table 1). The summary of observed response of Wall 6-7 is presented below:

- More extensive concrete spalling was observed for Walls 6 and 7 than for the previous five experiments. The reason for this behaviour was attributed to the larger axial load applied to

Walls 6 and 7 and the lower height to length ratio of these two wall panels, which resulted in a larger compression force acting at the wall panel toes.

- The extensive concrete damage at a drift level of 1.5% caused the outermost metal duct to be exposed and subsequently be pulled out of the wall panel as the connection splice failed , as shown in Fig. 8a.
- The wide cracks and more extensive concrete spalling observed in Wall 6 demonstrated the larger contribution of the wall panel flexural deformation to the overall response of Wall 6 in comparison with the previous five tests.
- Concrete spalling occurred in smaller area in Wall 7 in comparison with Wall 6 due to both the confinement reinforcement that ensured that the splice was maintained and the increased vertical reinforcement that supported the confinement reinforcement, as shown in Fig. 8b. Testing concluded at a drift level of 1.5% after the outermost connection reinforcement fractured at both ends of the wall panel.

Comparison with monolithic shear walls

All seven tested wall panels experienced less damage than expected for conventional monolithic reinforced concrete walls, and most of the damage was concentrated in the connection region. Excluding panel uplift, the width of cracks in the panel was less than 4 mm in all experiments, and concrete crushing was much less than typically observed for comparable (lightly reinforced) flexure controlled reinforced concrete walls (Lu et al. 2016). This difference was more obvious in Wall 4 because of the larger wall panel flexural strength in comparison with its connection strength, which is typical of jointed precast panel designs.

The effect of connection confinement

The influence of connection confinement on wall panel behaviour was found to be significantly dependent on the magnitude of compression stress at the wall panel toe. When the compression stress was large enough to cause substantial concrete spalling at the corners of wall panels, the use of confining

stirrups improved the connection performance and prevented spalling around the metal duct and associated degradation of the splice between the wall panel and the connection reinforcement, leading to the full capacity of the connection being achieved. In contrast, no significant difference in performance was observed when the confining stirrups were used in wall panels that had a smaller wall panel length and no additional axial force applied.

Force-displacement behaviour

The resultant force-displacement hysteresis responses for Wall 1 to Wall 4 are shown in Fig. 9. The behaviour of the four wall panels was very similar as their connection characteristics and subsequent failure modes were the same. During the first three forced-controlled loading cycles the wall panels were effectively elastic, whilst in the fourth cycle a nonlinear response was observed due to the commencement of both panel cracking and yielding of the wall-foundation connection reinforcement. In the following cycles a gap opened at the bottom of each of the four tested wall panels, causing pinching of the force-displacement response. This behaviour was due to the gap opening in the connection zone which decreased the stiffness of the wall panels because the moment was carried by only the connection reinforcement. Finally, all four tests were concluded when the connection reinforcement fractured, causing rapid strength degradation. The four wall panels had similar maximum lateral strengths of approximately 53 kN, and in all four wall panels the maximum measured lateral force was larger than the calculated nominal strength of the panel-foundation connection which was equal to 46 kN for Wall 1 to Wall 3, and 47 kN for Wall 4. All four wall panels achieved a drift capacity of 2% prior to fracture of the connection reinforcement.

The obtained force-displacement response for Wall 5 is shown in Fig. 10a. The wall panel behaved linearly during three force-controlled cycles, and in the next cycle yielding of the connection reinforcement commenced. At larger drift levels the hysteretic response began to display nonlinear behaviour, with larger residual displacement due to wall panel cracking and plastic deformation of the connection reinforcement. The magnitude of residual displacement was less than that measured during testing of Walls 1-4 due to fewer cracks forming in the wall panel and smaller gap opening at the

connection, attributed to the increased magnitude of applied axial load. When the 2.0% drift level was applied to the wall panel the lateral force reached the maximum recorded magnitude of 110 kN and at the next drift level of 2.5% more extensive cracking and concrete spalling occurred, leading to a reduction in wall panel stiffness, with the lateral force dropping to a magnitude of 75 kN (68% of the peak strength). Testing was concluded by applying a 3.5% drift level that caused fracture of the connection reinforcement. This reinforcement fracture occurred at a larger drift level than in the previous four experiments due to the reduced extent of gap opening at the connection and consequently smaller strains in the connection reinforcement for a given drift level. The maximum lateral force measured in this test was approximately twice the magnitude measured in the previous four experiments due to the larger axial load applied to the wall panel. The lateral strength of Wall 5 was also larger than the corresponding calculated lateral force for nominal flexural strength of the wall panel connection (87 kN). The other difference between this experiment and the earlier wall panel tests was that the unloading curve was less steeply inclined than for the four previous tests, causing smaller residual displacement at each cycle. In addition, there was less pinching of the force-displacement diagram for Wall 5 than for the previous four experiments. The reason for the differences in hysteretic response of Wall 5 in comparison to the previous four experiments was again attributed to the application of axial compression to the wall, which helped to close the wall-to-foundation joint when unloading.

In Fig. 10b and Fig. 10c the force-displacement response of Wall 6 and Wall 7 are shown. Larger lateral strengths were measured in these wall panels when compared with the previous five tests, due to the greater dimensions of the wall panels and the larger value of the axial load. At drift levels below 1.5% the behaviour of Wall 6 and of Wall 7 were almost identical, with peak lateral strengths reaching similar magnitudes of 308 kN for Wall 6 and 307 kN for Wall 7, although these peak strengths occurred at differing drift levels of 1.5% for Wall 6 and of 1.0% for Wall 7. The reason for the different drift capacity between the two experiments was attributed to the different failure mode in each experiment. Failure of Wall 6 was due to progressive concrete spalling which resulted in the metal duct becoming detached from the wall panel, as shown in Fig. 8a. In contrast the failure of Wall 7, which had more

robust splices between the connection reinforcement and the vertical reinforcement of the wall panel, was due to fracture of the connection reinforcement.

The back-bone force-displacement responses extracted from the first cycle to each drift level for each of the seven experiments are compared in Fig. 11. In order to eliminate the effects of the different wall panel lengths and heights the ratios of measured lateral force to calculated nominal strength of the connections were plotted versus the magnitude of drift level, to reveal that the backbone curves of all wall panels were similar. It was also found that the last three tested wall panels, where axial load was applied, had larger margins above the normalized nominal strength than for the other four experiments.

Deformation components

The contributions of the four in-plane mechanisms consisting of rocking, sliding, flexure, and shear deformation were measured in all experiments by using displacement gauges that were positioned on the wall panels and their connections to the foundation. Flexural deformations were obtained by measuring the rotation of the wall panel horizontal cross sections as proposed by Hiraishi (1984) and shear deformations were obtained using diagonal displacements, again based upon a previously proposed method (Hiraishi 1984) which considered the influence of flexural deformation on the measured diagonal displacements. The extent of wall panel rocking was calculated according to the uplift measured by two displacement gauges positioned at the extreme edges of the wall panel and sliding was obtained directly by a displacement gauge and an LVDT that were installed at the middle of the wall panel connection with the foundation. The sum of the four displacement mechanisms was compared with the measured displacement at the top of the wall panels and the difference between these two values indicated the measurement error, which was less than 10% for all seven tests. The contributions of each force-displacement mechanism to the overall response of the seven wall panels are shown in Fig. 12 and Fig. 13.

In general, the response of Walls 1-4 was dominated by rocking at the wall-to-foundation interface, contributing to between 69-90% of the total lateral displacement during larger drift cycles. The flexural deformation of Wall 1 was approximately 15% of the overall displacement when the connection

reinforcement was below the yield stress, but at larger drift levels yielding of the connection reinforcement resulted in a gap opening at the connection that facilitated rocking and sliding of the wall panel. Sliding displacements contributed more than flexural displacements to the total response of the wall panel at larger drift levels. Shear deformation had a negligible contribution to the force-displacement behaviour of Wall 1. A similar behaviour was observed for Wall 2 and Wall 3 but the contribution of rocking in Wall 4 was larger than for the previous three experiments because Wall 4 had a greater thickness and greater vertical reinforcement content, which limited both the flexural and shear deformations of the wall panel.

The contribution of rocking to the overall response of Walls 1-4 was larger than for Walls 5-7. The reason for this response was that the absence of applied axial load for Walls 1-4 facilitated rocking and limited the extent of concrete spalling, consequently decreasing the flexural deformation of these wall panels when compared to Walls 5-7. The contributions of sliding and rocking in Wall 5-7 were lower due to the application of axial load.

In Wall 6 and Wall 7 the flexural deformations of the wall panels had a greater contribution in comparison to the behaviour observed in the previous five tests. The increased flexural deformations were attributed to the increased panel dimensions and increased axial load, which resulted in increased panel cracking and spalling. Wall panel flexural deformation provided the largest contribution to the lateral displacement for Wall 6, whilst in Wall 7 panel rocking was more dominant. The larger contribution of flexural deformation in Wall 6 correlated with the observed increase in panel crack widths and spalling when compared to Wall 7 where the panel and connection splice remained less damaged and instead the wall panel rocked about the wall base.

Energy dissipation

In the displacement-based seismic design method the determination of equivalent viscous damping (EVD) is required. The EVD can be calculated by:

$$\xi = \frac{A_h}{2\pi\Delta_m F_m} \quad (1)$$

where ξ is equivalent viscous damping, A_h is the enclosed area of each cycle of the force-displacement diagram, Δ_m is the maximum displacement in a cycle, and F_m is the maximum lateral force in a cycle. The calculated EVD for the first cycle of each drift level for all wall panels is shown in Fig. 14. All wall panels had lower EVD when compared with commonly adopted values for monolithic reinforced concrete walls, which are typically greater than 20% at the failure cycle for wall panels subjected to moderate level of axial load and greater than 25% for wall panels having no applied axial load (Lu et al. 2016, Zhang and Zhihao 2000). The reason for this reduced level of EVD, when compared with comparable monolithic construction, was attributed to the reduced extent of plastic deformations, which mostly developed in the connection zone rather than as distributed plasticity associated with a traditional plastic hinge region at the base of the wall. The EVD increased as greater drift levels were applied to the wall panel up to a drift level of 1.5%, because at larger drift levels a larger extent of plastic deformation of the connection reinforcement and more extensive concrete cracking occurred than at smaller drift levels. In all experiments the EVD reached a peak value at a drift level of 1.5% and then reduced as larger drift levels beyond 1.5% were applied to the wall panels. This behaviour was mainly because of greater pinching of the force-displacement diagram at larger cycles that was attributed to a combination of bond slip and plastic deformation of the connection reinforcement as a gap opened between the wall panel and the foundation.

Wall 1 to Wall 4 had larger EVD than for the other three tested wall panels because no axial load was applied to these walls, resulting in a fatter hysteresis response. Walls 5-7 had lower EVD than for the previous four experiments because the applied axial load resulted in a more steeply inclined unloading force-displacement curve and an associated reduction in the enclosed area of each cycle. The EVDs of Wall 6 and Wall 7 were approximately the same when the applied drift level was below 1.5%, but the EVD of Wall 6 reduced at higher drifts due to splice degradation and disconnection of the two outermost reinforcing bars that caused a reduction in the capacity of the wall panel to dissipate energy.

CONCLUSIONS

The cyclic response of precast concrete wall panels with grouted metal duct connections representative of those commonly used in existing buildings was examined, focusing on parameters such as applied axial load level, wall panel geometry, and varying detailing of the splice confinement between the wall panel and connection reinforcement. The following conclusions were drawn:

- The overall behaviour of all seven wall panels was consistent with the design philosophy for precast concrete panels having equivalent monolithic connections. The wall panel stiffness and nonlinear behaviour were consistent with that of monolithic concrete walls.
- The measured lateral strength of all wall panels was larger than their calculated nominal strength, and in most cases the full capacity of the connection was achieved prior to failure. Despite the low drift capacity of some walls, the test results confirmed that walls having metal duct connections in existing buildings that were designed for nominally ductile actions are likely to have adequate seismic strength.
- Due to the increased ratio of panel strength to connection strength, the behaviour of the wall with a double layer of reinforcement was dominated by rocking about the wall base, with no cracking in the panel itself. Rocking was found to contribute less to lateral deformation for the wall panels having a single layer of reinforcement because of flexural deformations due to panel cracking.
- For short wall lengths and low axial loads, panel failure was controlled by fracture of the connection reinforcement. For the longer wall panel with larger axial load and no confinement reinforcement, concrete spalling resulted in the metal duct becoming detached from the wall panel when the compression strain at the wall panel compression toe was large enough to cause extensive concrete spalling. This type of failure is more likely to occur in wall panels with a larger length, greater connection reinforcement content, and a greater magnitude of applied axial load.

- The measured compression strain at the wall panel toes prior to spalling was approximately 0.002, which is significantly less than typical concrete failure strains. This small failure strain highlights the lack of robustness of singly reinforced walls with no confinement reinforcement.
- The use of transverse reinforcement in the form of stirrups to confine the connection reinforcement and a larger quantity of vertical reinforcement around metal ducts increased the strength and ductility of the wall panel toe, limiting the concrete spalling and preventing failure of the metal duct connection by reinforcement pull-out. However, the use of confinement stirrups did not increase the drift capacity of the wall panels, with the failure mode shifting to reinforcement fracture due to concentrated rocking about the wall base. In the walls where spalling of the wall panel toe did not occur, the influence of the stirrups on the connection performance and overall behaviour of the wall panels was insignificant.

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Table 1. Details of the test wall panels

Wall panel number	Length (mm)	Height (mm)	Aspect ratio	Thickness (mm)	Connection reinforcement	Vertical reinforcement	Confining reinforcement	Axial Load (% $A_g f'_c$)
1	1000	3000	3	150	HD16@400	Single layer HD12@225	-	0
2	1000	3000	3	150	HD16@400	Single layer HD12@225	Spiral	0
3	1000	3000	3	150	HD16@400	Single layer HD12@225	Rectangular	0
4	1000	3000	3	200	HD16@400	Double layer HD12@225	-	0
5	1000	3000	3	150	HD16@400	Single layer HD12@225	-	5%
6	2000	4000	2	150	HD16@450	Single layer HD12@225	-	5%
7	2000	4000	2	150	HD16@450	Single layer HD12@225	Rectangular	5%

Table 2. Properties of utilized materials (all stresses in MPa units)

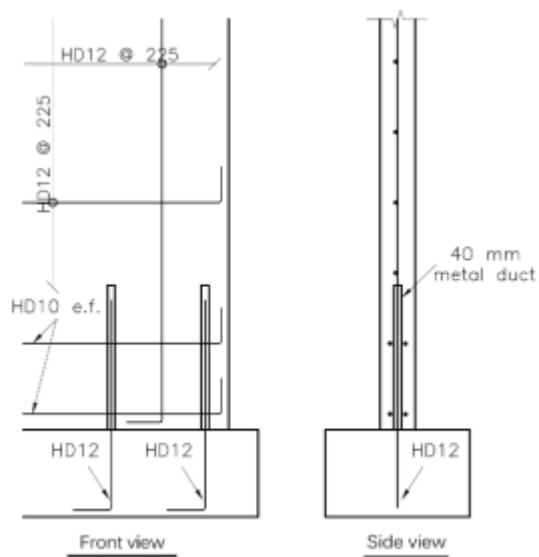
Wall panel number	Grout strength	Concrete strength	Connection reinforcement			Panel reinforcement		
			Yield stress	Ultimate stress	Strain at peak strength	Yield stress	Ultimate stress	Strain at peak strength
1	58	46	473	632	0.10	523	653	0.11
2	43	54	473	632	0.10	523	653	0.11
3	56	46	473	632	0.10	523	653	0.11
4	50	56	473	632	0.10	523	653	0.11
5	52	43	482	629	0.11	520	641	0.11
6	54	53	482	629	0.11	520	641	0.11
7	64	45	482	629	0.11	520	716	0.12

Table 3. Summary of observed crack widths and failure drifts

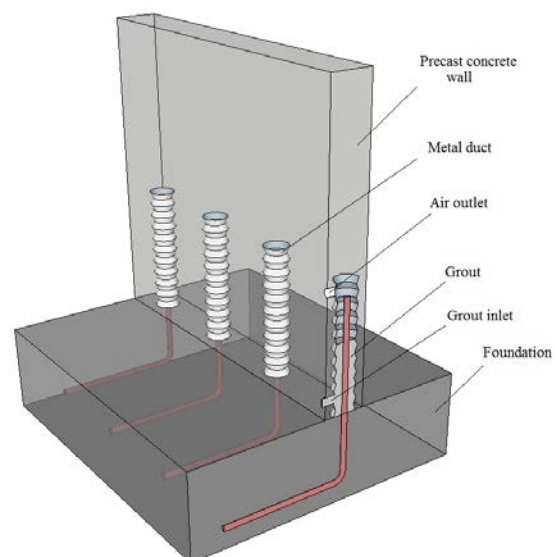
Wall panel number	Maximum wall panel crack width (mm)	Corresponding drift at maximum crack width (%)	Failure drift (%)	Maximum crack width at failure* drift (mm)	Wall panel uplift at failure* drift (mm)
1	1.4	2	1.8	1.0	25.0
2	1.6	2	2.1	1.0	25.0
3	0.4	2	2.0	0.2	27.2
4	0	2	2.0	0	25.6
5	1.8	1.5	2.1	0.4	17.8
6	4.0	1.5	1.5	0.5	**
7	3.0	1	1.2	1.4	19.2

* Failure was defined as drift at which either metal duct pull-out or reinforcement fracture occurred.

** Wall panel uplift could not be measured due to extensive spalling.



(a) An example of a conventional metal duct wall-to-foundation connection detail



(b) Grouted metal duct connection (wall panel and foundation reinforcement not shown)

Fig. 1. Grouted metal duct connection details.



Fig. 2. An example of metal duct connection damage during the 2010/2011 Canterbury earthquakes.

(Photo credit: Ken Elwood)

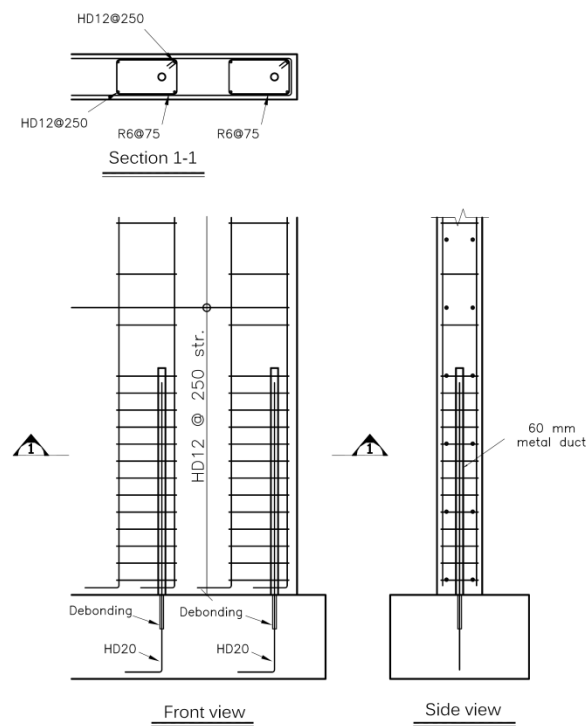
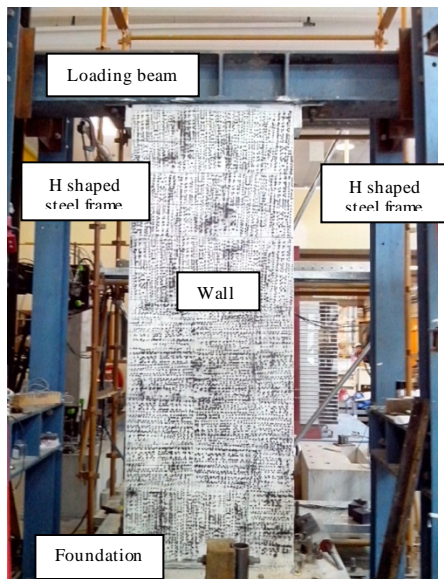
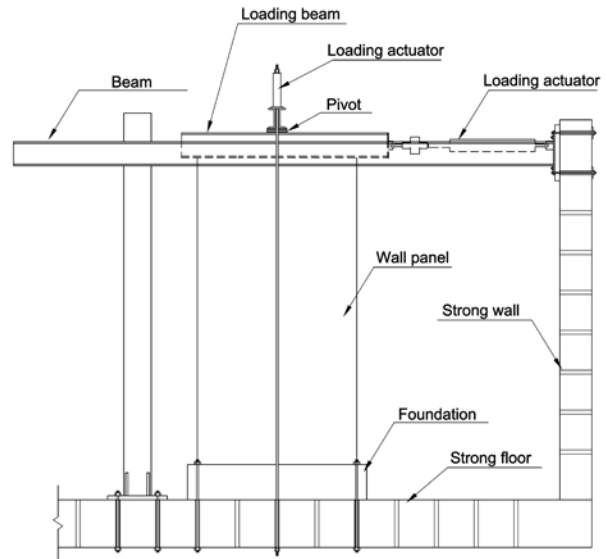


Fig. 3. An example of SESOC recommended detailing (SESOC 2013) for metal duct connections.



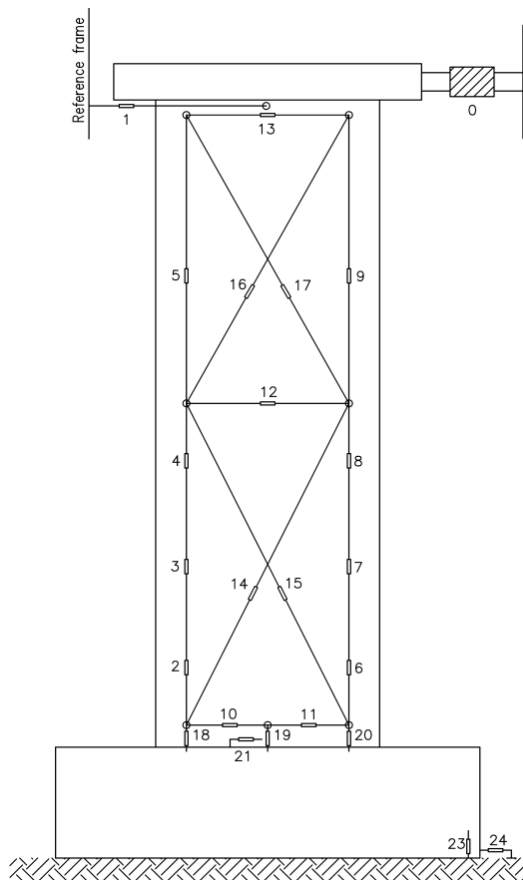
(a) Test setup without application of axial load



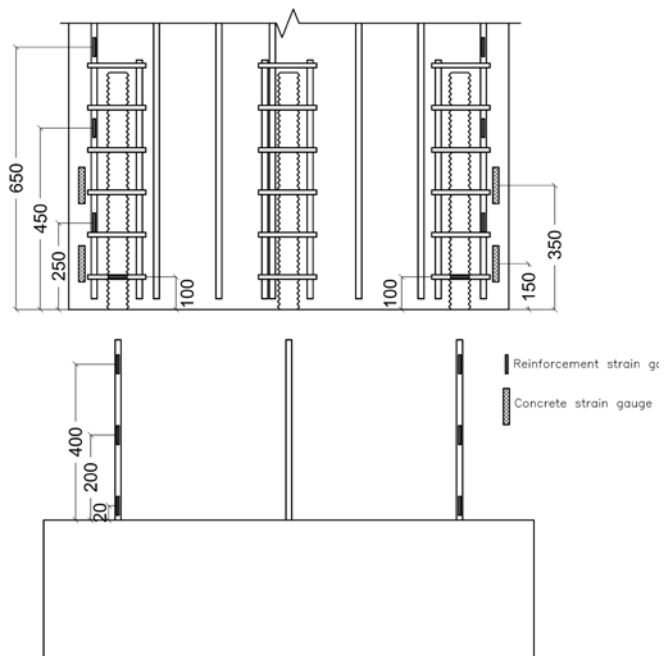
(b) Test setup with application of axial load

503

Fig. 4. Test setup



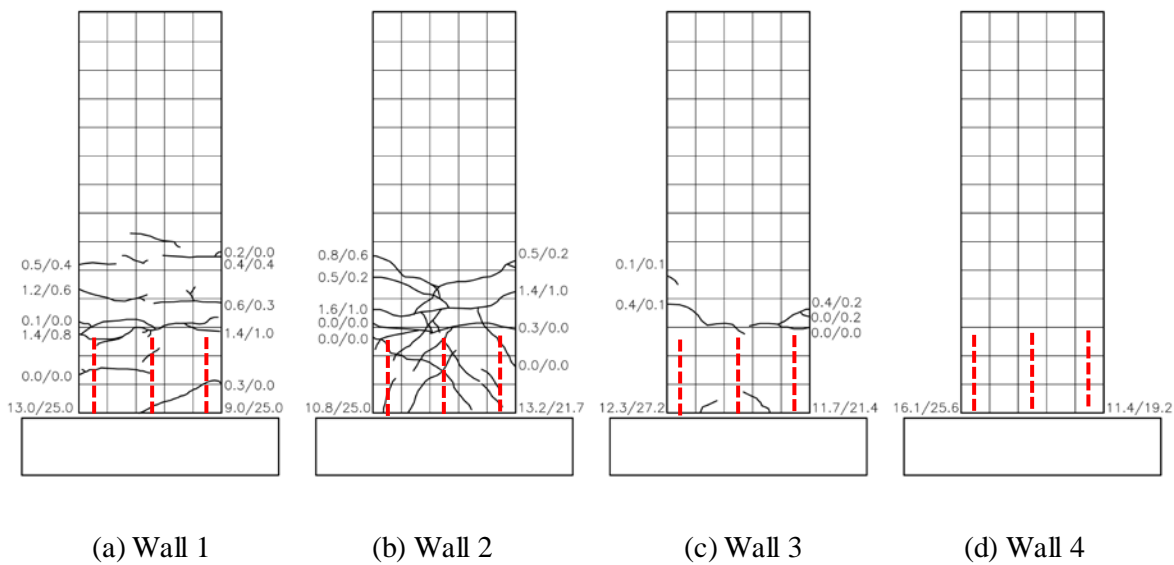
(a) Non-embedded instrumentation



* Not to scale; horizontal reinforcement not shown for clarity

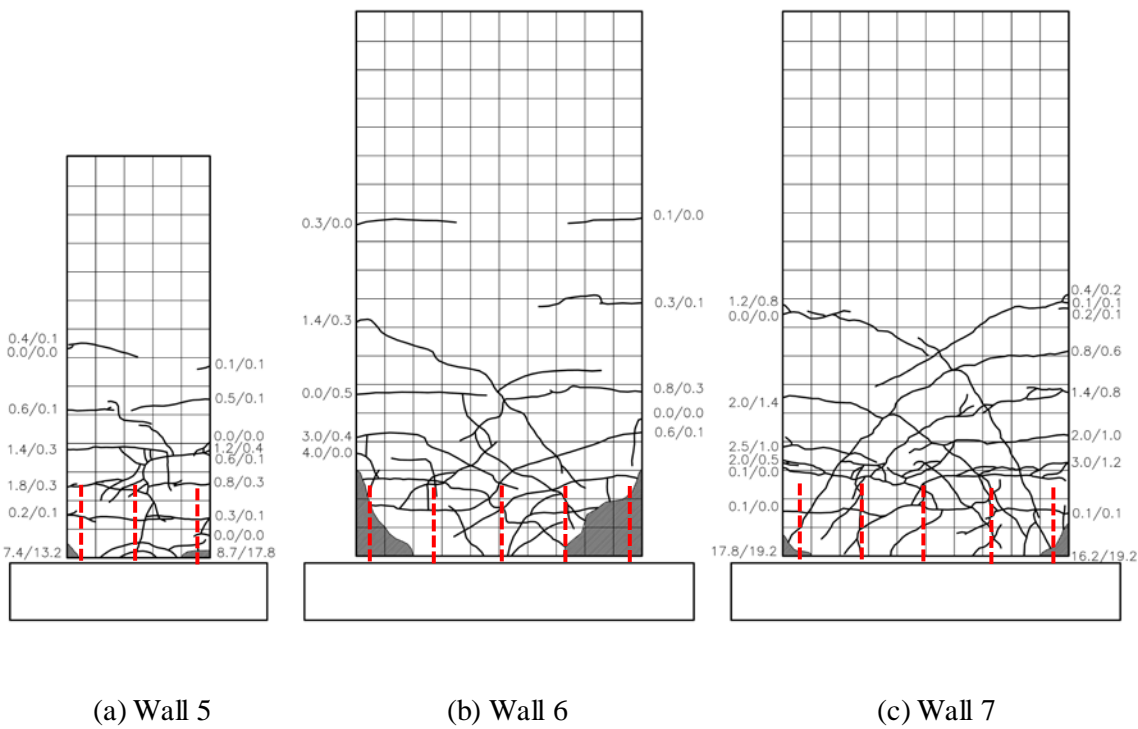
(b) Distribution of embedded strain gauges

Fig. 5. Distribution of instrumentation (front view of wall panel)



* The connection reinforcement is shown with a dash line.

Fig. 6. Crack patterns for wall panels without application of axial load (dimensions refer to crack width at maximum load/crack width at last cycle)



* The top of connection reinforcement is shown with a dash line.

507 **Fig. 7.** Crack patterns for wall panels with application of axial load (dimensions refer to crack width
508 at maximum load/crack width at last cycle)

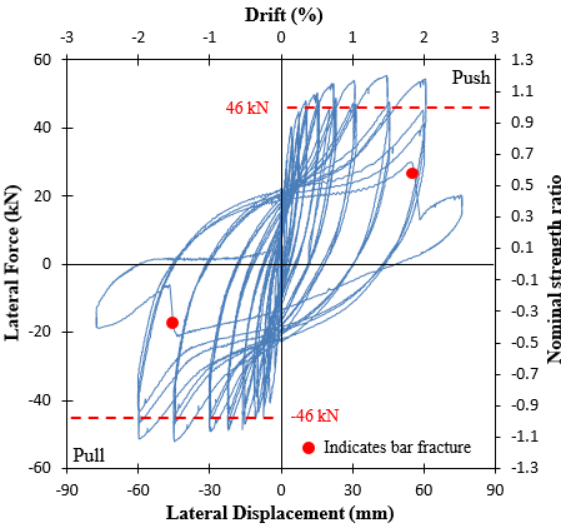


(a) Wall 6

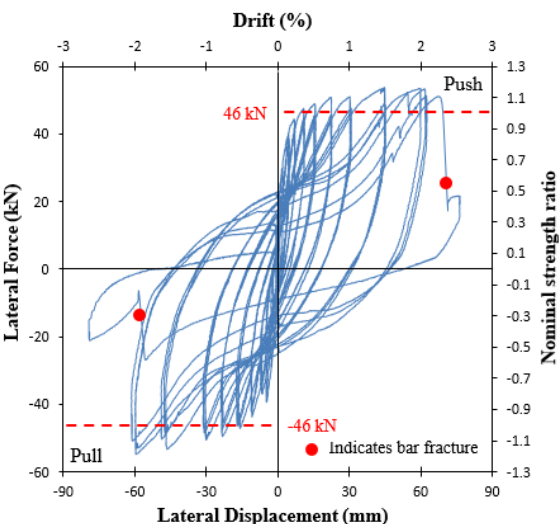


(b) Wall 7

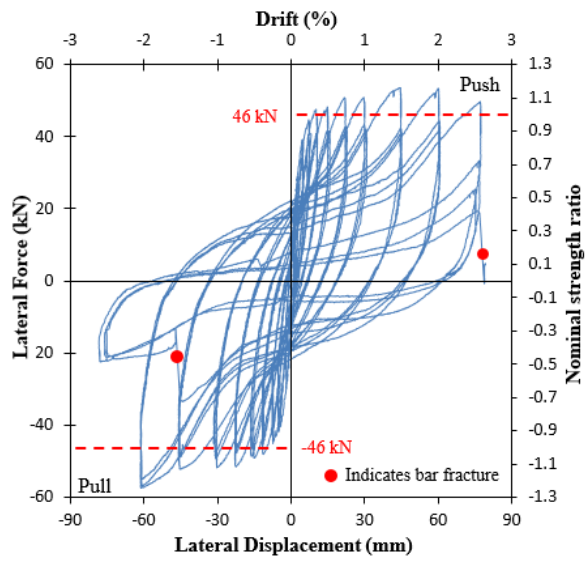
509 **Fig. 8.** Condition of compression zones of Wall 6 and Wall 7 at the conclusion of testing



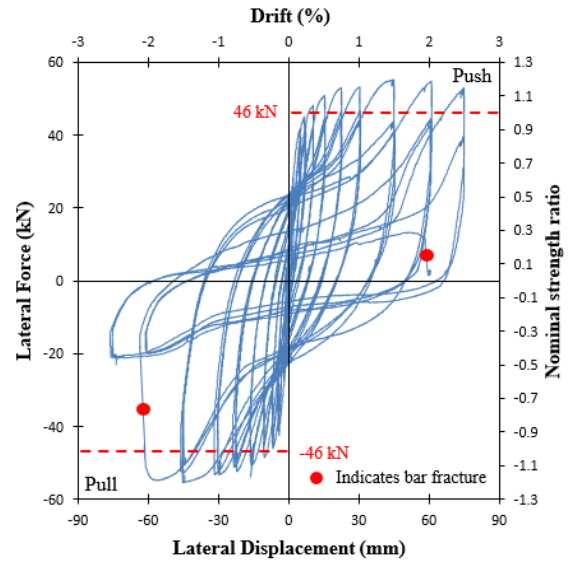
(a) Wall 1



(b) Wall 2



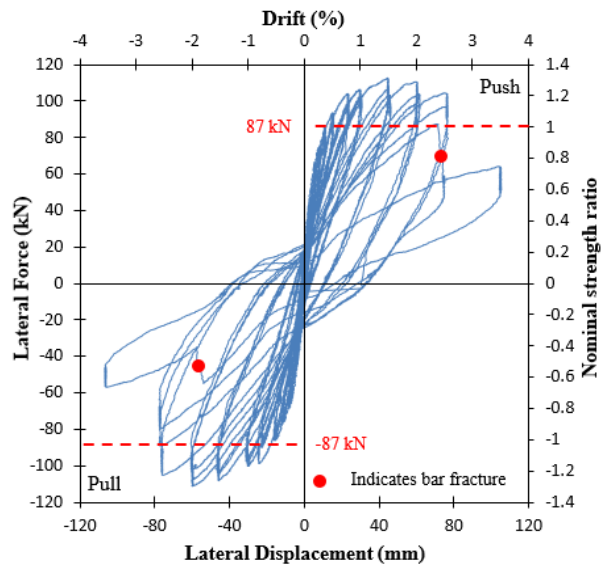
(c) Wall 3



(d) Wall 4

510

Fig. 9. Hysteresis response of wall panels with no applied axial load



(a) Wall 5

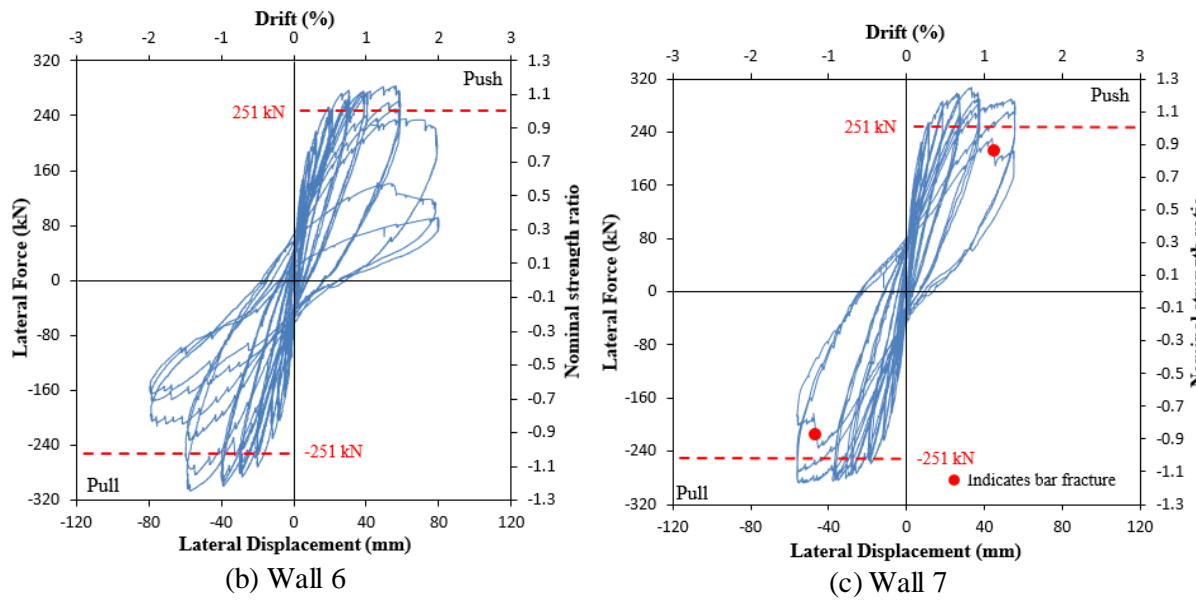


Fig. 10. Hysteresis response of wall panels with applied axial load

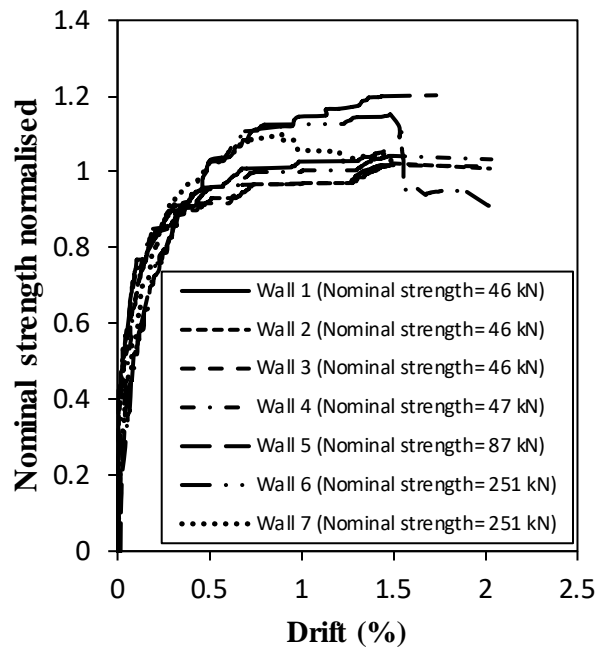
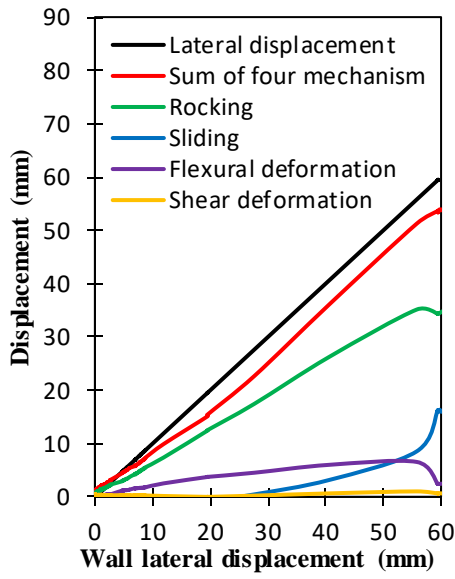
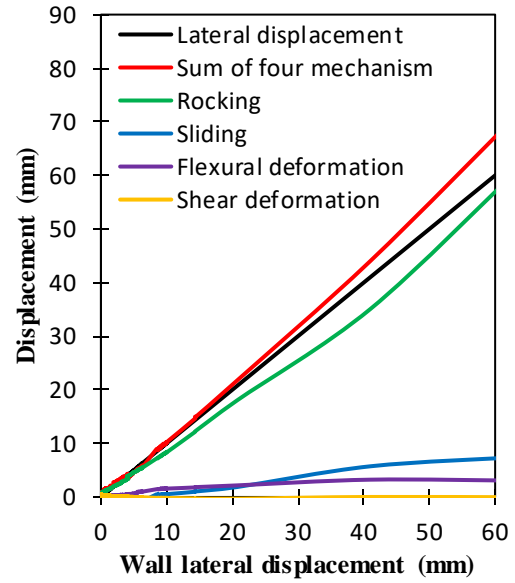


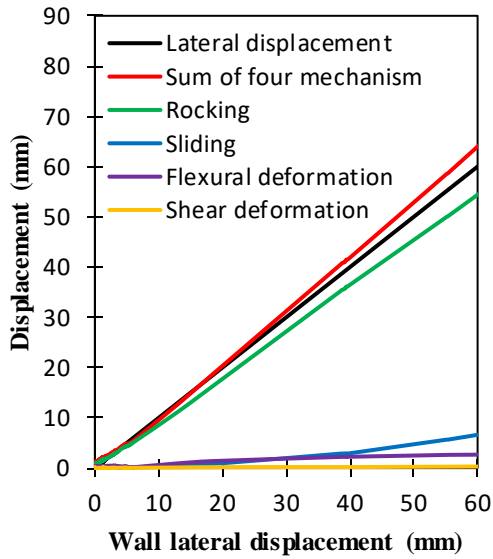
Fig. 11. Force-displacement backbone curves normalised against calculated nominal strength



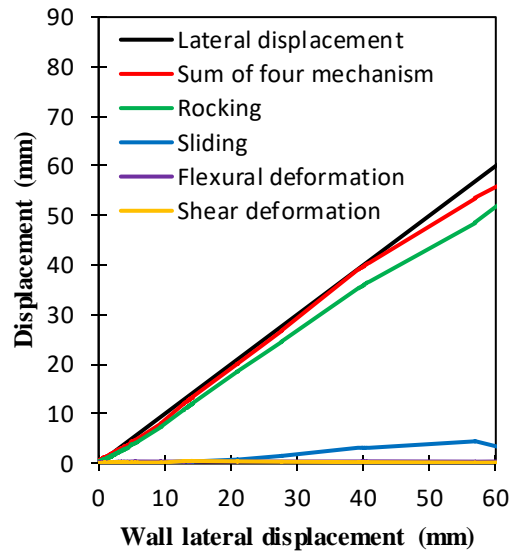
(a) Wall 1



(b) Wall 2



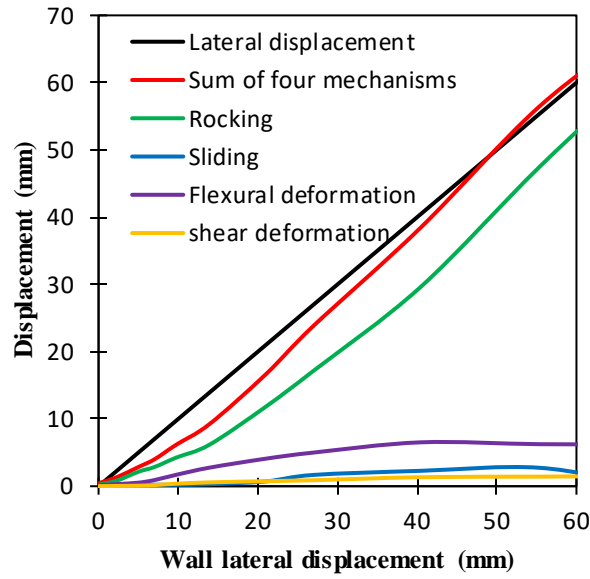
(c) Wall 3



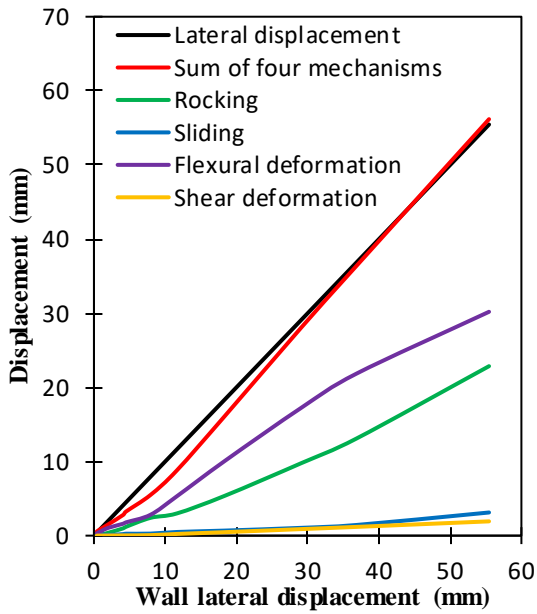
(d) Wall 4

Fig. 12. Contribution of each deformation mode to overall response for wall panels with no applied

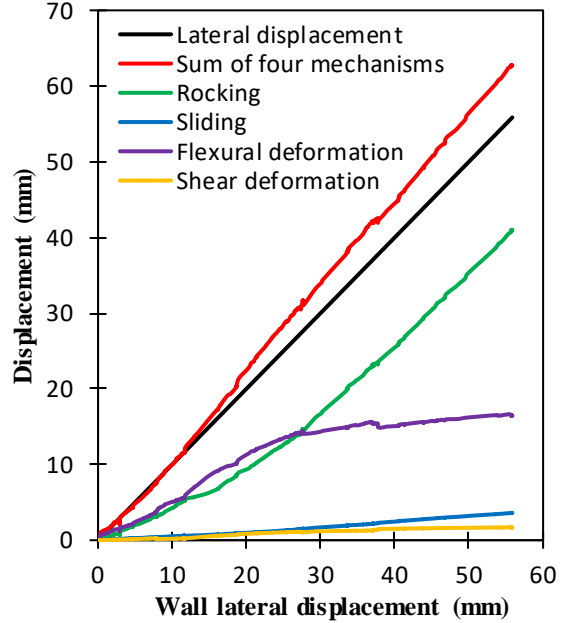
axial load



(a) Wall 5



(b) Wall 6



(c) Wall 7

Fig. 13. Contribution of each deformation mode to overall response for wall panels with applied axial load

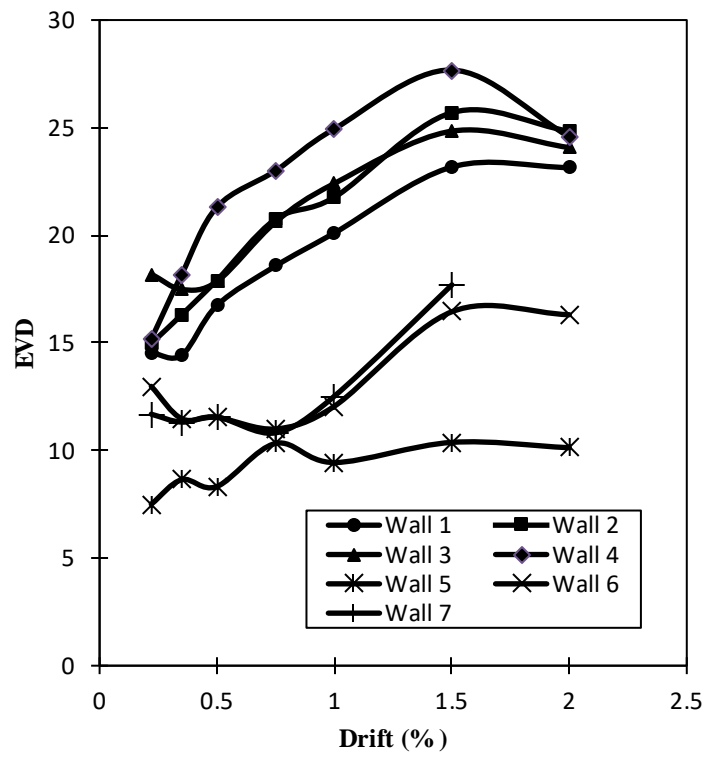


Fig. 14. Equivalent viscous damping of each wall panel