A New Rocking Concrete Shear Wall with Self-Centring Friction Connections

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ABSTRACT

In recent years, the intention towards developing and utilising low-damage systems in seismic resisting structures has been increasing amongst researchers and engineers. Replaceability and repairability after earthquakes even greater than the Ultimate Limit State (ULS) design level, more reliability against aftershocks, and being immediately occupiable after severe earthquakes are some of the advantages of these systems. Different types of rocking concrete shear walls have been developed and used as low-damage structural systems. In this paper, a new low-damage rocking concrete shear wall has been introduced, developed and tested. Self-centring friction dampers are used as wall to base connections to control the rocking motion in the proposed system. First, the conceptual framework and the analytical procedure to design each system in detail and predict the performance are presented. Afterwards, an experimental program is designed and conducted to verify the analytical procedure developed. Later, the numerical investigation and digital modelling of the rocking wall system is presented. The numerical outcome is also compared to the test results and the models are verified accordingly. Finally, in order to assess the seismic performance of the system within a structure, a Non-Linear Time History (NLTH) analysis is carried out on a single-degree-of-freedom prototype structure. The selected response parameters are compared to the code requirements and the outcome of NLTH analysis is discussed.

1 INTRODUCTION

Ductile concrete shear walls dissipate earthquake input energy mostly through the yielding of longitudinal reinforcements in flexure [1]. Other minor sources of energy dissipation are also observed such as yielding of transverse reinforcement, concrete crushing, aggregate inter-lock friction and sliding shear [2]. However, these behavioural modes are not desirable due to associated brittle failure modes. In general, ductile concrete shear walls have performed well in saving lives, the main purpose that they were designed to, in the past earthquakes [3]. Providing strength, stiffness, ductility and energy dissipation are the good seismic resisting characteristics of Reinforced Concrete (RC) walls [4]. The plastic hinge region of the RC walls should be detailed adequately in order to represent the repetitive cyclic damping and to achieve the desired ductility [5]. However, ductile detailing is complex and requires strong engineering and construction skills and experiments.

In some cases, unexpected failure modes such as local instability, bar rupture and web crushing have been observed in the experimental studies and past earthquakes (Fig. 1) [6, 7]. Residual deformation is expected in RC shear walls after moderate to severe earthquakes. This residual displacement along with the damages in concrete and reinforcements imposes extensive repair or reconstruction costs [8]. Therefore , researchers suggested a move from ductile shear walls towards rocking shear walls as damage avoidance systems [9, 10].

(a)

(b)

(c)

(d)

Figure 1. Observed failure modes of structural walls during past earthquakes: (a) Bar yielding followed by bar rupture, (b) Toe crushing, (c) Diagonal shear, (d) Compression failure and buckling [11].

End users and authorities expect resilient structures. A resilient structure has negligible structural and minor non-structural damage after a severe earthquake. Researchers introduced Damage Avoidance Design (DAD) philosophy to achieve resilient structures [9]. DAD aims to achieve a repeatable and damage avoidance cyclic energy dissipation in addition to zero or negligible residual deformation. Therefore, the building could be occupied immediately after earthquakes and is ready to withstand aftershock without any deterioration in the structural characteristics [12].

Rocking concrete shear walls have been developed as a damage avoidance solution for DAD [13-17]. In rocking concrete shear walls, traditional reinforcement yielding and concrete crushing are not expected [13, 18-20]. Unbonded post-tensioning [10, 18, 24, 128, 148] and self-centring dampers [111] have been used to control the rocking motion and to provide controlled self-centring capability. Therefore, problems related to damage and residual deformation are almost overcome in rocking walls designed based on DAD philosophy.

The concept of rocking motion has always been a point of interest for seismic researchers and engineers [25, 174-179]. During the past earthquakes, it has been observed that the structures which unexpectedly rocked around their base suffered from less damage compared to those of fixed base [25]. Therefore, several different systems incorporating rocking motion have been introduced for earthquake resistant structures. The rocking motion has been incorporated in both the foundation level [180-182] and throughout the structure. At the structure level, different concepts and structural materials have been used. For example, steel rocking braced frames [183-185], steel rocking shear walls [186-188], timber [189, 190] and concrete [35, 148, 191] rocking walls. Different structural configurations have also been developed to implement the rocking motion in the structures. Single rocking walls [148], coupled rocking walls [189, 192] for increasing the capacity and multi rocking walls [193] to decrease the higher mode effects in high-rise buildings can be named as some examples. Along with these new rocking systems, several design procedures have been developed [194, 195].

The most common way of implementing a controlled rocking behaviour is to use post-tensioning (PT) cables along with supplemental yielding dampers [23]. However, other types of energy dissipaters such as friction or viscous dampers have also been used in the PT rocking wall structures [35]. It should be noted that using on-site post-tensioning causes construction complexity and additional labour. Also, yielding dampers should be inspected after moderate to severe earthquake and repaired or replaced if required. Unbonded post-tensioning was used in pre-cast concrete walls in PREcast Seismic Structural Systems (PRESSS) in the early 1990s [31]. Detailed analytical studies on the behaviour of Single Rocking Wall (SRW) systems were conducted by Kurama et al. [149]. They suggested using additional spiral reinforcement at the wall toes to mitigate the damage of concrete due to crushing. Kurama and colleagues [33] also performed a series of studies on the unbonded PT walls with additional supplemental damping such as yielding, friction or viscous damping. The results showed that the additional dampers effectively reduce the seismic demand if they are properly designed. Priestley [24] used U-shaped Flexural Plates (UFP's) in coupled rocking wall systems. Other efforts were made for using internal grouted mild steel reinforcement for additional hysteresis damping [28, 29]. Externally mounted mild steel, Tension and Compression Yielding (TCY) dampers, as well as viscous dampers, were designed and tested by Marriot et al. [35]. Sritharan and colleagues [34] suggested a PT rocking wall system as Pre-cast Wall with End Columns (PreWEC).

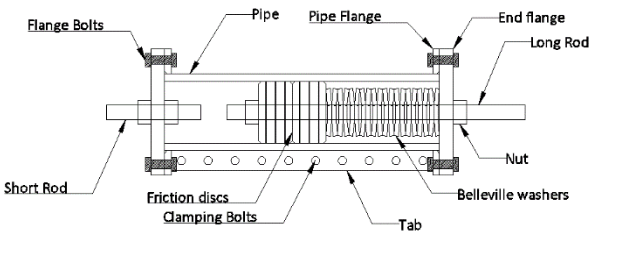
Considering the previous research related to the rocking wall structures, it is observed that a limited effort [185, 196] has been made in developing rocking walls with self-centring dampers. These devices provide self-centring and energy dissipation in one compact connection [58, 155] and can be used for controlling the rocking motion. The use of two self-centring dampers, the Resilient Slip-Friction Joint (RSFJ) [59] and Self-centring Structural Connector (SSC) [153], in rocking concrete walls, is investigated in this study. Then, the performance of the system is examined via large-scale cyclic testing. Also, a numerical modelling approach is introduced and verified by the experimental results. Finally, the seismic performance of a prototype structure designed based on this new rocking system is examined using NLTH analysis and compared to the code requirements.

2 CONCEPT DEVELOPMENT AND ANALYTICAL INVESTIGATION

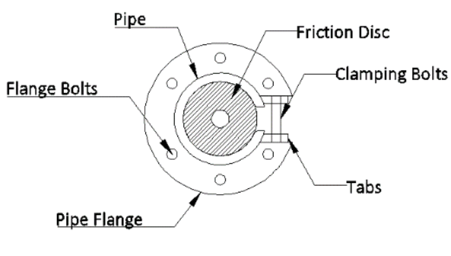
Self-centring Structural Connector (SSC)

In this section, the performance and configuration of SSC are discussed [163]. Different components of SSC are shown in Fig. 2.

Disc springs are responsible for the self-centring capability. Every single spring has a specific displacement capacity, Δ_s, and a corresponding load capacity, F_s, at the maximum displacement, which is called flatness load. It should be mentioned that the stack of the springs should be pre-stressed to the desired pre-stressing force before being inserted inside the tube.



SSC Longitudinal Section



SSC Transverse Section

Figure 2. Self-centring Structural Connector (SSC).

The friction of the internal surface of the tube with friction rings is the source of energy dissipation. At this stage, the tube clamping bolts are pre-stressed to the required clamping force. This force creates the normal force (perpendicular to the surface) between the disc and the tube for friction.

In this section of the paper, the force-displacement performance of the SSC is analyzed. As can be seen from the Fig. 3, the SSC has a flag-shaped force-displacement curve.

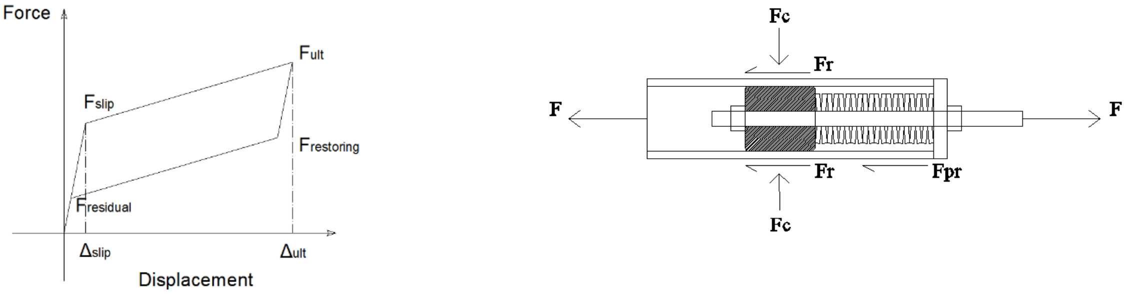


Figure 3. Free body diagram and force-displacement response.

At the slip stage, friction force ($F_{friction}$) and the pre-stressing force ($F_{prestressing}$) inside the spring resist against the movement. F_{slip} can be obtained using Eq. 1:

$$F_{slip} = F_{friction} + F_{prestessing} \tag{1}$$

The calculation of Δ_{slip} using the mechanics of materials rule can be easily done. When the discs are flat, the damper has theoretically reached its maximum force capacity (F_{ult}). F_{ult} can be estimated using Eq. 2.

$$F_{ult} = F_{friction} + F_{prestessing} + K\Delta = F_{friction} + F_s \tag{2}$$

where K is the equivalent stiffness of the set of springs and Δ is the displacement capacity of the springs after pre-stressing. F_s represents the force capacity of the springs. The restoring force, $F_{restoring}$, can be calculated using Eq. 3.

$$F_{restoring} = F_s - F_{friction} \tag{3}$$

Eventually, the displacement reaches zero. The external residual force in the system at this stage, $F_{residual}$, can be obtained using Eq. 4:

$$F_{residual} = F_{prestressing} - F_{friction} \tag{4}$$

In order to obtain the self-centring condition, the pre-stressing force of the spring must be greater than the friction force.

$$F_{prestressing} \geq F_{friction} \tag{5}$$

In the design of SSC, the normal force acting on the surface between the piston and the cylinder can be adjusted by the external clamping force. This force (F_c) is applied using the tube clamping bolts and nuts and can be calculated using Eq. 6.

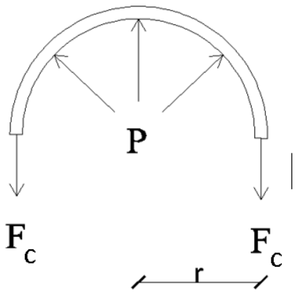


Figure 4. Free body diagram of a cylinder with internal pressure.

As shown in Fig. 4, considering the equilibrium equations for the longitudinal section of the cylinder, the relationship between the clamping force (F_c) and the pressure (P) on the tube can be obtained using Eq. 6 for a cylinder with the radius of r and the length of L.

$$F_c = PrL \tag{6}$$

At this stage, the friction force (F_{fr}) can be calculated using the pressure on the tube (P) and the friction coefficient, μ.

$$F_{fr} = 2\pi\mu F_c \tag{7}$$

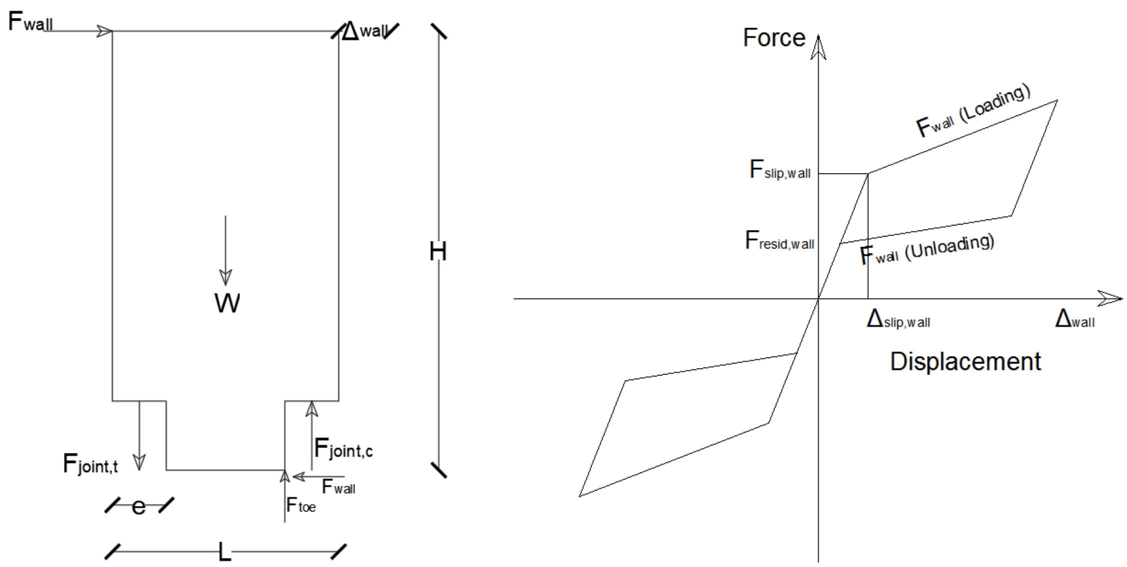
The maximum displacement of the springs stack after pre-stressing is equal to the maximum displacement capacity of the damper, Δ_{ult}, which can be calculated using Eq. 8.

$$\Delta_{ult} = n\Delta_s(1 - \frac{F_{pr}}{F_s}) \tag{8}$$

The reader is referred to (ref) for more details on the design and performance of the SSC.

3 LOAD-DISPLACEMENT PERFORMANCE OF ROCKING WALLS

First, the performance of rocking walls with double-acting connections are assessed and the equations are developed. Second, the same assessment will be done for rocking walls with tension-only connections. The free-body diagram of the system with double-acting connections is mentioned in Fig. 5(a).



(a) **(b)**

Figure 5. a) Free-body diagram of rocking walls equipped with double-acting self-centring connections and b) load-displacement response of the wall.

Considering the free-body diagram of the wall at maximum lateral drift (Fig. 5(a)) and the load-displacement performance of the wall mentioned in Fig. 5 (b), the performance parameters can be calculated as follows by taking moment around the wall toe. At the slip level, when joints have reached their slip force, the wall starts to rock.

$$F_{slip,wall} = \frac{1}{H}\left[F_{slip,joint}(L-e) + w\frac{L-2e}{2}\right] \tag{9}$$

Where, $F_{slip,wall}$ is the lateral load acting on the wall at the onset of rocking motion, $F_{slip,joint}$ is the connections' slip force and W is the weight of the wall. Other geometric parameters are mentioned on the diagram (Fig. 5 (a)).

After the onset of rocking movement, the tension and compression joints contribute to transfer the loads to the foundation based on their lever arm to the wall toe. The moment equilibrium at this loading stage results in Eq 10.

$$F_{wall} = \frac{1}{H}\left[F_{joint,t}(L-\frac{3e}{2}) + w\frac{L-2e}{2} + F_{joint,c}\frac{e}{2}\right] \tag{10}$$

F_{wall}, is the lateral force acting on the wall during the loading and unloading stages of rocking motion, $F_{joint,t}$ is the force in the connection under tension and $F_{joint,c}$ is the force in the connection under compression. The residual force in the wall ($F_{resid,wall}$) can be obtained by replacing $F_{slip,joint}$ by $F_{resid,joint}$ in Eq. 9.

A relationship between $F_{joint,t}$ and $F_{joint,c}$ can be derived according to the displacement compatibility at the base by considering the "plane section remain plain" assumption. the wall as a rigid element. The displacement compatibility can be expressed by Eq. 11 which after being combined with the load-displacement performance of the joints, the relationship between the tension and compression forces for both loading and unloading stages can be presented (Eq. 12 and Eq. 13, respectively).

$$\frac{\Delta_{joint,t}}{L-\frac{3e}{2}}=\frac{\Delta_{joint,c}}{\frac{e}{2}} \tag{11}$$

$$F_{joint,c}=\frac{e}{2L-3e}\left(F_{joint,t}-F_{slip,joint}\right)+F_{slip,joint} \tag{12}$$

$$F_{joint,c}=\left(\frac{e}{2L-3e}\right)\left(F_{joint,t}-F_{resid,joint}\right)+F_{resid,joint} \tag{13}$$

$\Delta_{joint,t}$ and $\Delta_{joint,c}$ are the joint displacement due to F_{wall} under tension and compression, respectively. $F_{resid,joint}$ is the residual force in the joint at the end of the unloading stage.

The wall top displacement is a combination of the wall rigid body rotation due to rocking and its elastic deformation. The top displacement of the wall can be calculated using Eq. 14 to Eq. 16.

$$\Delta_{wall}=\Delta_{elastic}+\Delta_{rot} \tag{14}$$

$$\frac{\Delta_{rot}}{H}=\frac{\Delta_{joint,t}}{L-\frac{3e}{2}}=\frac{\Delta_{joint,c}}{\frac{e}{2}} \tag{15}$$

$$\Delta_{elastic}=\frac{F_{wall}H^3}{3EI} \tag{16}$$

$\Delta_{elastic}$ is the elastic deformation of the wall due to F_{wall}. E and I are the modulus of elasticity and moment of inertia of the wall section.

The same analogy can be used to develop the equations governing a rocking wall with tension-only connections. The free body diagram of such a system is illustrated in Fig. 6(a). There is a main difference between tension-only and double-acting systems. In tension-only systems, the governing equations are the same for before and after the onset of rocking motion (Eq. 17). As for the displacements, the same equations as those developed for the double-acting systems can be used.

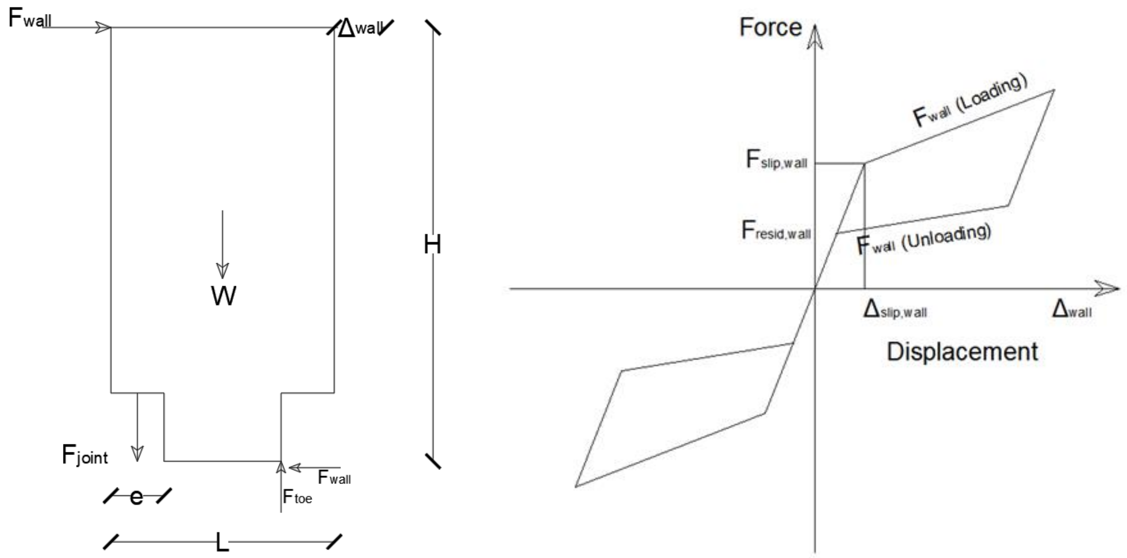


Figure 6. The free-body diagram of rocking walls equipped with tension-only self-centring connections.

$$F_{wall}=\frac{1}{H}\left[F_{joint}\left(L-\frac{3e}{2}\right)+w\frac{L-2e}{2}\right] \tag{17}$$

$$\frac{\Delta_{rot}}{H} = \frac{\Delta_{joint,t}}{L - \frac{3e}{2}} \tag{18}$$

The developed equations will be verified through experimental investigation presented in the upcoming sections.

4 PROPOSED CONCEPT

The proposed solution not only resolves the above-mentioned challenges but also has the advantage of optimising the amount of reinforcement required within the concrete panel. Also, this concept is suitable for prefabricated construction and promotes the use of pre-cast concrete in seismic resisting structures.

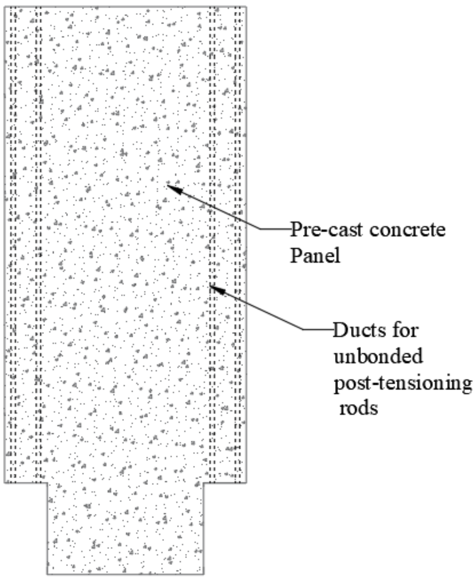
Pre-stressing has always been a well-accepted method to control the cracking and deflection in reinforced concrete. Pre-stressed and post-tensioned members have widely been used in the construction industry. The concept of off-site pre-stressing and/or post-tensioning of prefabbed reinforced concrete is not common for seismic resisting elements. Even though on-site unbonded post-tensioning is used for concrete rocking systems, the design complexity and the extra on-site work required for post-tensioning is one of the challenges from both design and construction points of view. In the proposed solution, the advantages of post-tensioning are used while on-site extra work is not required as the post-tensioning can be done in the factory.

In this concept, as shown in Fig. 7, a concrete panel that is already post-tensioned in the factory using unbonded rods is transferred to the construction site and then mounted and connected to the foundation using the self-centring hold-downs. It should be noted that the post-tensioning aims to prevent the wall from cracking and therefore an internal post-tensioning of the wall prior to mounting will suffice. This means that there is no need to use unbonded PT cables to connect the wall to its foundation. It is also not required to use complex connection detailing or a highly reinforced panel with special seismic detailing. In most cases, a minimum required reinforcement could be enough. The details of the developed concept are presented as follow.

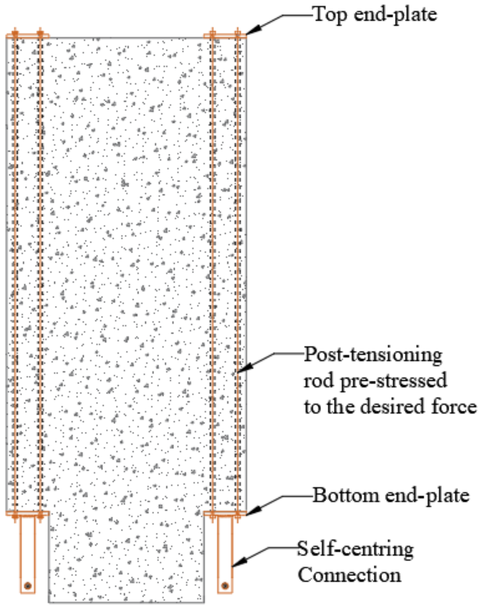
As shown in Fig. 7, firstly, the concrete panel should be post-tensioned internally using unbonded rods up to a certain pre-stressing force level. The post-tensioning force can be applied using a rod or cable inserted through the ducts in the boundary regions of the wall and connected to the endplates (Fig. 7). In the next section, the concept is described in more details through designing the concrete panel used for the experimental investigations.

In order to eliminate the tension cracks, a pre-stressing concept is developed. In this concept, the pre-cast concrete panel is connected to the foundation using the self-centring dampers. This precast concrete panel is post-tensioned using unbonded cables or rods. After manufacturing and considering proper timing for the concrete curing, the pre-cast concrete panel will be compressed using the unbonded tensioning elements. Post-tensioning can be done either at the factory or the construction site when the wall is laid down on the floor. This decreases the construction time and costs in comparison to the current on-site post-tensioning concepts. The wall can then be mounted vertically and get connected to the foundation using the brackets.

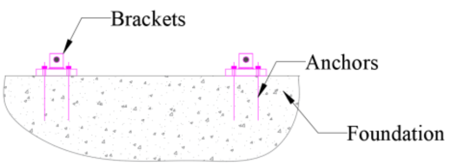
Another advantage of this system in comparison to the current post-tensioned rocking walls is that there is no need to design the post-tensioning elements for high displacement demands as the flexibility of the system comes from the dampers rather than the PT tendons.



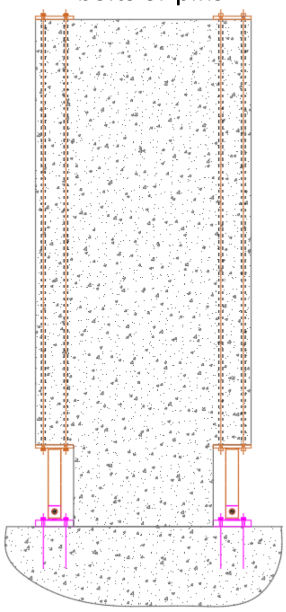
Phase 1: In the Factory
Pre-cast concrete panel



Phase 2: Either in the factory or on-site
Post-tensioning rods added and pre-stressed with:
arrangement 1, self-centring connections already welded to the bottom endplate
or,
arrangement 2, Self-centring connections to be connected to the bottom endplate on-site using bolts or pins



Phase 3: On-Site
Brackets anchored to the foundation



Phase 4: On-Site
Wall mounted and connected to the brackets

Figure 7. Different components and construction phases of the proposed rocking pre-cast concrete wall system.

5 EXPERIMENTAL INVESTIGATION

An experimental program is designed and conducted in order to verify the performance of the developed system. The details of the experimental program are presented in this section.

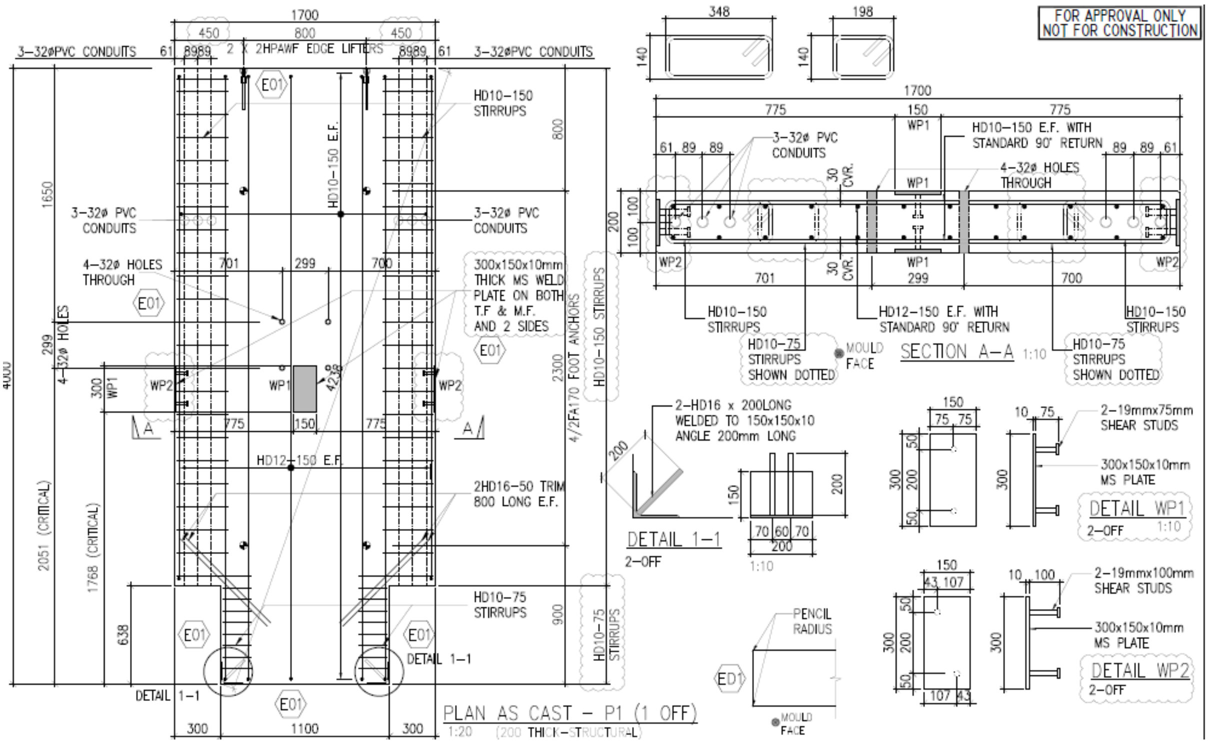
5.1 Wall Panel Design

The required equations for designing the wall were developed in the previous section. The details of the designed concrete wall specimen are presented in Fig. 8(a). The wall dimensions are summarised in Table 1. The wall was designed in a way that the maximum force induced in the self-centring joints were limited to 200 kN. The reason for selecting the 200 kN limit for designing the joints was the capacity of the available Universal Testing Machine (UTM) in the lab. The maximum drift of the wall at the ultimate force was considered to be 2.5% as per the recommendations of New Zealand Standard (NZSEE) [164] for the Ultimate Limit State (ULS) earthquakes. Accordingly, by knowing the external forces and dimensions of the wall, the internal forces can be calculated to design the wall. The wall has been designed using the procedure developed in the previous section. The reinforcement shop drawings and the wall reinforcement before pouring the concrete are shown in Fig. 8.

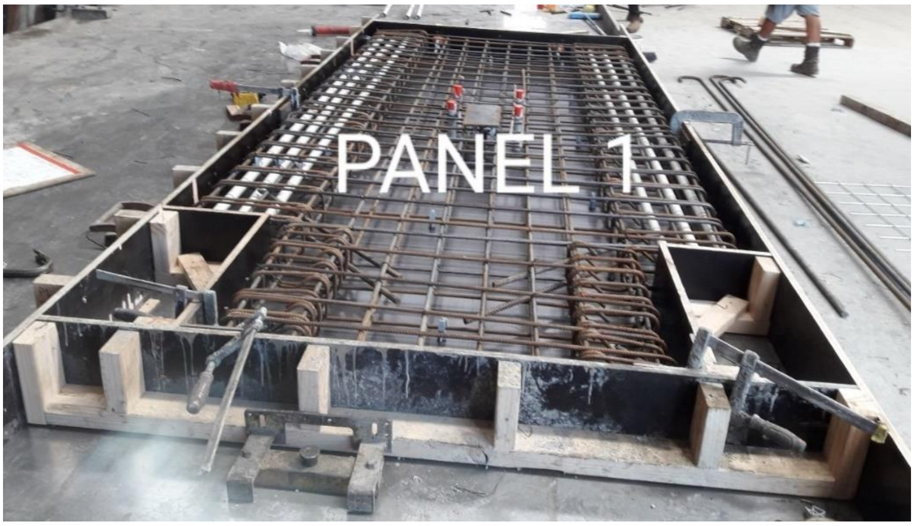
Table 1. Design parameters of the tested wall.

L (mm)	1700
e (mm)	300
H (mm), Actuator level	2190
Total Height of Wall (mm)	4000
T (mm), Wall thickness	200
W (kN)	35350
F’c (MPa), concrete compressive strength	40
Fy (MPa), reinforcement yielding strength	500
E (MPa)	31600

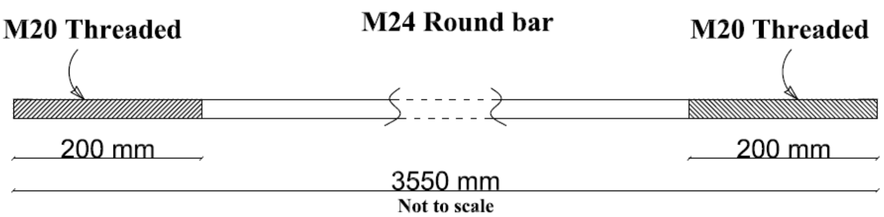
As can be seen from Fig. 8(b), three ducts are located at each boundary of the wall in order to pass the post-tensioning rods through them. The post-tensioning rods are size M24 Grade 8.8 threaded at the ends to M20 (Fig. 8(c)).



(a)



(b)

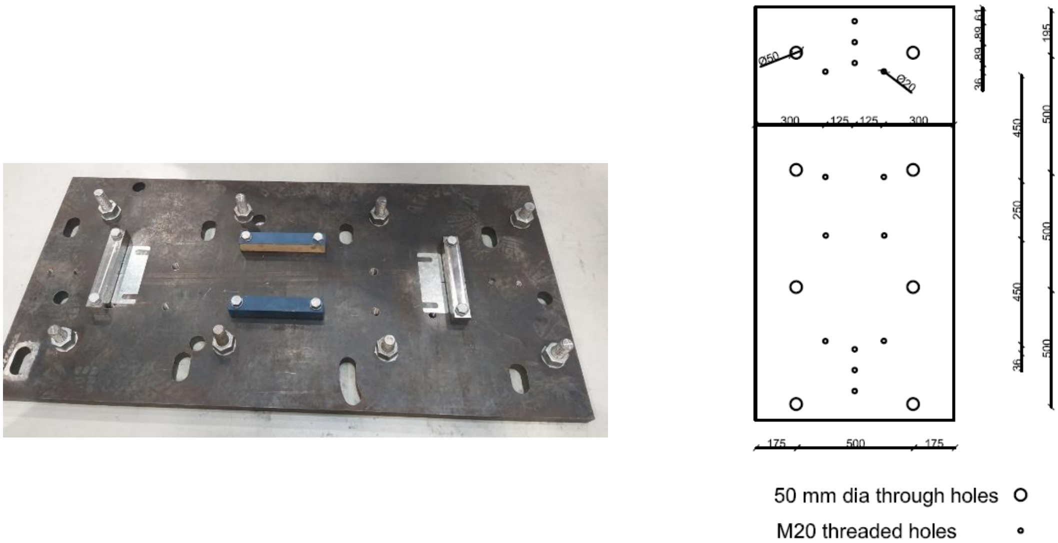


(c)

Figure 8. Concrete panel reinforcement: a) shop drawings, b) reinforced wall before the concrete pour, c) post-tensioning rod.

5.2 Test setup

The details of the test setup and components are described in this section. As two different concepts have been tested, even though the wall and general setup were similar for both concepts, different details were considered for each test. As shown in Fig. 9(a), a mild steel base plate with a thickness of 50 mm has been used for connecting the wall and joints to the strong floor. The base plate is tightened to the strong floor using pre-tensioned high strength bolts. The amount of the pre-tensioning force should be enough to achieve a sliding friction resistance between the foundation and the strong floor. The sliding resistance should be more than the shear force acting on the contacting surface. The base plate drawings are illustrated in Fig. 9(b). The location of the required plain holes (for the base plate connection to the floor), as well as the threaded holes (for the hold-downs connection to the base plate), are mentioned on the drawing.



(a) **(b)**

Figure 9. Rocking wall base plate: a) base plate connected to the strong floor and b) arrangement of the holes.

Lateral support has been used in order to maintain safety and to restrain the wall in the out-of-plane direction. The lateral support consists of four columns located at the corners of the base plate and two tie-beams (Fig. 10). Timber spacers have been placed between the wall and the tie beams to prevent the wall from out-of-plane movement. The contacting surface between the wall and timber spacers were lubricated with grease to eliminate the friction between the surfaces. As can be seen in Fig. 10, the beams were tied together using the straps in order to increase the overall stiffness of the set-up.

The lateral load was applied to the wall with an actuator equipped with a built-in load cell. The height of the actuator from the top of the steel base plate was 2190 mm. The push and pull forces were transferred from the actuator to the wall using the endplates with dimensions of 660*660*60mm (Fig. 10). The loading endplates were connected using four high strength M36 rods. The actuators were mounted close to the middle of the wall (rather than at the top) to facilitate the connection of the actuator to the specimen and the strong wall.

As shown in Fig. 11, two timber blocks have been used as out-of-plane shear keys. These shear keys prevent the wall from out-of-plane movement and twisting around its axis resulting from actuator imperfections.

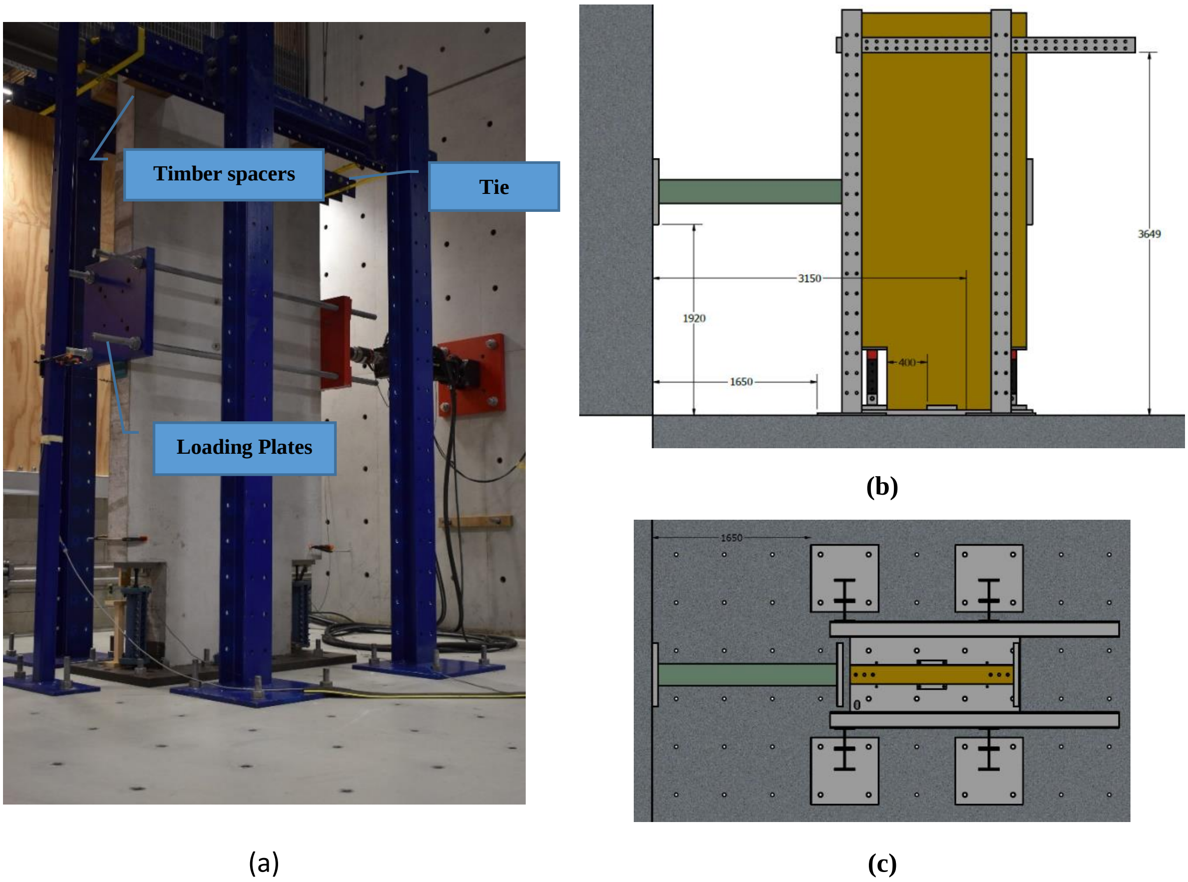


Figure 10. Lateral support: a) Test setup, b) schematic elevation view, c) schematic plan view.

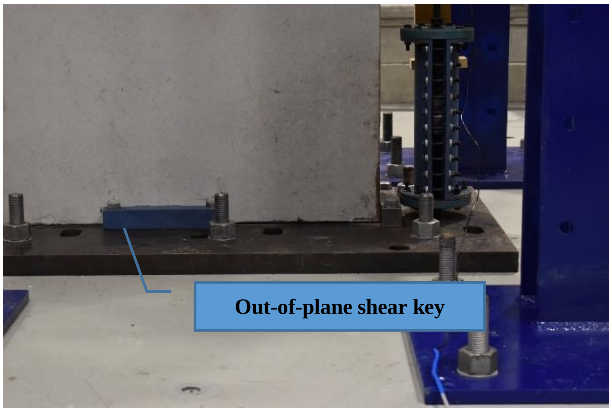


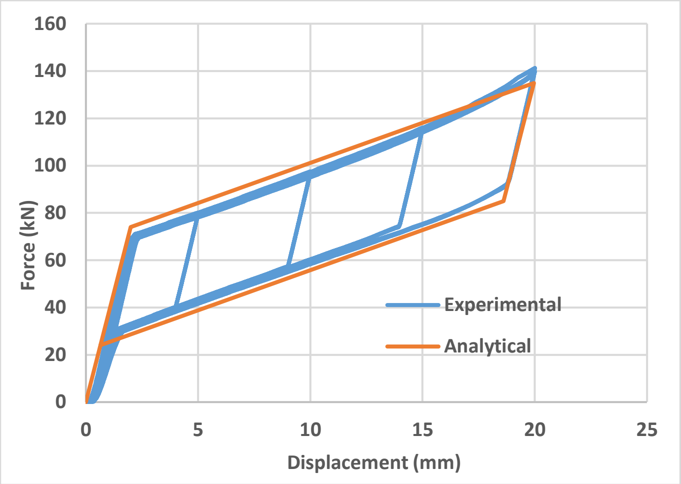
Figure 11. Out-of-plane shear keys.

6 SSC SHEAR WALL

The SSC joints used for the rocking wall tests were originally designed for the proof of concept of the joints [163]. Later, the same joints were used for the rocking concrete shear wall test. The joints are designed for an ultimate force capacity of 135 kN and a maximum displacement of 20mm. The design parameters of the SSC are mentioned in Table 2. The SSCs were tested using the UTM machine to verify the load-displacement performance of the joints. The assembled joint in the UTM is shown in Fig. 12. The load-displacement response obtained from the test is compared to the one predicted by the analytical equations verifying the accuracy of the developed model given the agreement between the results.

Table 2. Design parameters of tested SSCs.

Δ_S (mm)	1.80
F_{pr} (kN)	49.00
F_S (kN)	110.00
n	20
Δ_{ult} (mm)	20.00
F_c (kN)	26.50
m	0.15
F_{fr} (kN)	25.00



(a) **(b)**

Figure 2. SSC component test: a) comparison of analytical and experimental performance, b) test setup.

In this section, the details related to the connection of the SSC to the wall and the foundation base plate is presented. Rod M20 Grade 10.9 has been used to connect the joint to the wall endplate and the foundation. As shown in Fig 13a and b, the wall endplate and the foundation base plate were tapped for the threaded holes required for the rod connection. The SSC needed to be connected to the wall first. This was done by turning the joint to screw it to the wall endplate while the wall was horizontally laid on the floor before mounting. When the wall was vertically mounted on the base plate, the SSC needed to be twisted around its axis to be screwed to the foundation base plate (Fig. 13(a)). There should be enough engagement length for the threads to maintain the desired capacity. At this stage, the nut at the top of the SSC was tightened to the endplate restraining the joint from free rotation around its axis. Later, the nut at the bottom of the joint was tightened to the base plate to release the joint in compression and to prevent any compression force to be generated in the joint. In fact, the tension-only performance of the SSC was achieved by loosening the bottom nut as shown in Fig. 13(b). Also, it allows the joint to rotate from its bottom side to be compatible with the rotation induced by the rocking motion of the wall (Fig. 13(c)). For the SSC shear wall, two high strength steel blocks have been used at the wall toes to resist the shear force. These shear keys are shown in Fig. 13(d).

The completed test setup for the SSC shear wall is shown in Fig. 14(a). The wall was tested under 5 full cycles of lateral drift up to 1.6% where the maximum capacity of the joints was achieved. The load-displacement performance of the tested wall with the SSC dampers are shown in Fig. 14(b). The results have shown that the developed system is capable of resisting lateral loading with a repetitive and predictive response. As can be seen from the graph, the actual performance of the wall is well predicted by the analytical model. The performance of the wall was also tested under high frequency reversed cyclic testing as shown in Fig. 14(c). A repetitive load-displacement performance was achieved for five full cycles with a loading frequency of 0.75 Hz. The wall, joints and their connection were observed thoroughly after the test and no significant damage was detected confirming the robustness of the rocking-wall system developed with self-centring friction dampers as hold-downs.

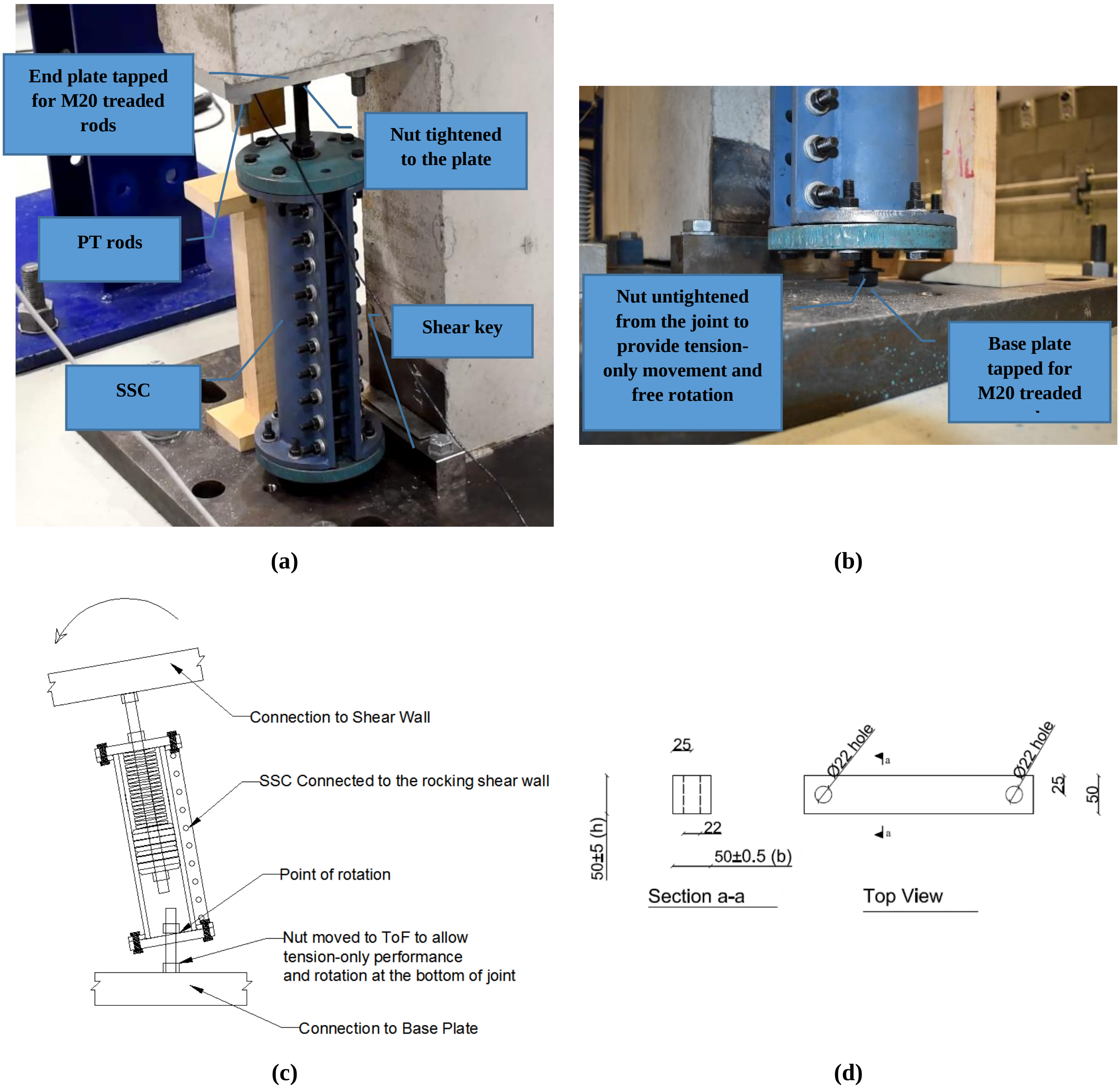
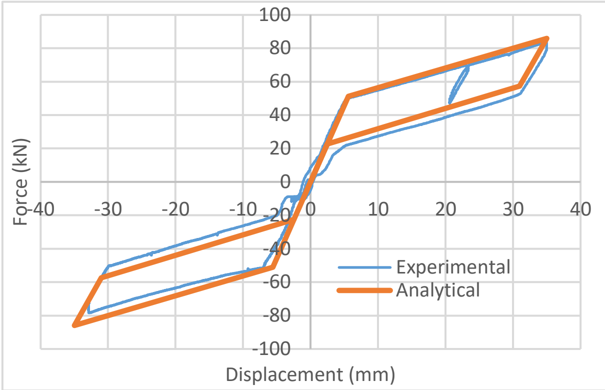
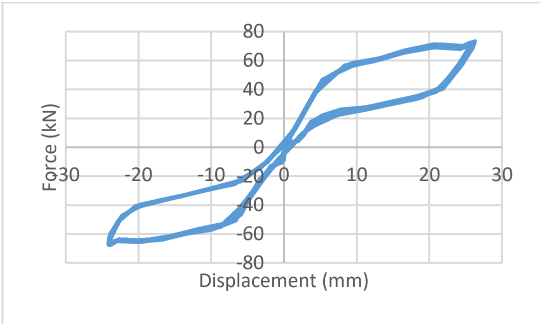


Figure 13. SSC connection to shear wall and foundation: a) connection at the top of the joint, b) connection at the bottom of the joint, c) rotation compatibility with rocking motion, d) shear keys



(a) **(b)**



(c)

Figure 14. SSC shear wall: a) test setup, b) comparison of analytical and experimental results, and c) results under high-frequency loading.

Note: displacements are inferred displacements at the actuator level based on draw-wire readings

7 DISCUSSION AND CONCLUSION

This study proposes a rocking shear wall system with self-centring friction dampers as an

alternative design to conventional fixed-base shear walls. Rocking shear walls can improve the seismic response of the structure considerably and mitigate the likelihood of irreparable damages.

It should be noted that an additional energy dissipation mechanism helps to improve the response of the rocking wall systems by reducing the drift demand. In addition, when additional damping is employed in a rocking wall, associated damage with pounding between the wall toe and foundation, such as toe concrete crushing, will be controlled effectively.

Self-centring friction dampers have been used in this study to provide the required energy dissipation mechanism. It is proposed to use the dampers at the wall base to connect the wall boundary to the foundation.

The proposed self-centring friction devices in this study provide repetitive and reliable energy dissipation and can be considered as robust elements for the damage avoidance design (DAD) system developed in this study.

In this section, the findings of this study are discussed as follows.

1. It can be observed from the experimental results that tension-only SSC and double-acting RSFJ rocking concrete shear walls tested in this study provide viable solutions for seismic damage avoidance design of structures. The repetitive cyclic behaviour under high-frequency cyclic loading confirmed the reliability of the proposed systems.

2. the analytical procedure developed for predicting the load-displacement performance of the rocking walls were proved to be accurate when compared to the experimental results. Overall, there is less complexity in designing rocking walls with tension-only dampers used as hold-downs compared to double-acting dampers. The analytical equations developed for predicting the performance of the dampers have shown good agreement with the experimental results.

3. It should be noted that when dampers are used as hold-downs, large bending moments are expected at the base of the wall. This bending moment results in highly reinforced sections at the base. Also, additional reinforcement is required for anchoring the dampers to the bottom of the wall. In addition, bending tension cracks decrease the stiffness of the wall significantly. Consequently, the effectiveness of the damping devices will be sacrificed as less deformation is to be transferred to the dampers due to wall flexibility as the result of cracking. A new pre-stressing method has been proposed in this study to overcome cracking related issues. In this concept, before mounting the rocking wall, the wall is internally pre-stressed before being mounted on the construction site. The benefits of the proposed pre-stressing method are as follows.

- Tension stresses and tension cracks are eliminated.

- Highly-reinforced sections are not required and usually, a minimum code reinforcement will suffice.

- There is no need to design and use embedded anchors for connecting the dampers to the wall. The end plates used for wall pre-stressing can be used for connecting the dampers to the wall.

- Complex and costly on-site unbonded post-tensioning, which is common for other rocking wall systems, is not required in the proposed system make it more efficient compared to the conventional rocking systems.

- the system can be optimised for mass production and modular construction as minimum site work is required for mounting and assembly.

4. A simplified numerical analysis with widely used commercial software was used to determine the level of the internal stresses within the wall. There has been a reasonable agreement between the proposed analytical procedure and the numerical results. Minor concrete spalling at the wall toe was observed during the test. Also, tension cracks were not detected within the wall after the test. These observations verify that the level of damage is within a DAD desired performance. Therefore, the system is proved to be a robust solution for DAD.

5. Nonlinear time-history analysis was carried out to evaluate the seismic response of the proposed system with double-acting connections. Three different scaled ground motions were used, and the maximum displacement response of the system was evaluated against the design objectives. The response in all three cases was less than the code (NZS 1170.5) specified requirement confirming the reliability of the proposed system.

6. For future studies, it is recommended to further assess the performance of the proposed system through large scale three-dimensional shake table tests. The effect of deformation compatibility with the gravity structure, bi-directional loading and concurrency effects should be studied throughout the shake table testing

program. More importantly, the effect of vertical accelerations on the behaviour of rocking structures should be studied experimentally as currently there is a lack of extensive study in this area.

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