Tenth U.S. National Conference on Earthquake Engineering
Frontiers of Earthquake Engineering
July 21-25, 2014
Anchorage, Alaska

SHAKE TABLE TESTING OF UNBONDED POST-TENSIONED PRECAST CONCRETE WALLS

Maryam Nazari[1], Sriram Aaleti[2] and Sri Sritharan[3]

ABSTRACT

Structural systems designed to current seismic codes are aimed at preventing collapse, thereby ensuring life safety, but they may sustain irreplaceable damage in well-designed plastic hinge regions. This design approach can also cause permanent lateral deformation to the structures as well as significant economic losses due to downtime. Consequently, buildings designed to meet life safety as the only seismic design objective has been found to be unsatisfactory. To address this concern, seismic resilient structural systems have been developed using unbonded post-tensioning as the primary reinforcement over the past two decades. Unbonded post-tensioned precast concrete rocking wall is one such structural system, designed such that the post tensioning strands in the wall remain elastic during design-level seismic events, helping the walls to re-center while experiencing little or no structural damage. This paper summarizes the response of four single rocking walls completed as part of an ongoing NEESR-CR project. The shake table tests were conducted on 1/3.6-scale rocking walls at the University of Reno at Nevada (UNR) NEES facility. The wall specimens were subjected to a series of short- and long-duration input motions. In addition to evaluating the dynamic test set up, the influence of several design parameters such as initial prestressing stress and area of strands on lateral response behavior of walls is presented in this paper. Using wall response parameters such as drift, concrete strain, accelerations and residual drift obtained from different earthquake ground motions, it is found that dependable seismic performance can be achieved for these rocking walls.

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Maryam Nazari, Sriram Aaleti, Sri Sritharan. Shake Table Testing of Unbonded Post-Tensioned Precast Concrete Walls. *Proceedings of the 10th National Conference in Earthquake Engineering*, Earthquake Engineering Research Institute, Anchorage, AK, 2014.

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Structural systems designed to current seismic codes are aimed at preventing collapse, thereby ensuring life safety, but they may sustain irreplaceable damage in well-designed plastic hinge regions. This design approach can also cause permanent lateral deformation to the structures as well as significant economic losses due to downtime. Consequently, buildings designed to meet life safety as the only seismic design objective has been found to be unsatisfactory. To address this concern, seismic resilient structural systems have been developed using unbonded post-tensioning as the primary reinforcement over the past two decades. Unbonded post-tensioned precast concrete rocking wall is one such structural system, designed such that the post tensioning strands in the wall remain elastic during design-level seismic events, helping the walls to re-center while experiencing little or no structural damage. This paper summarizes the response of four single rocking walls completed as part of an ongoing NEESR-CR project. The shake table tests were conducted on 1/3.6-scale rocking walls at the University of Reno at Nevada (UNR) NEES facility. The wall specimens were subjected to a series of short- and long-duration input motions. In addition to evaluating the dynamic test set up, the influence of several design parameters such as initial prestressing stress and area of strands on lateral response behavior of walls is presented in this paper. Using wall response parameters such as drift, concrete strain, accelerations and residual drift obtained from different earthquake ground motions, it is found that dependable seismic performance can be achieved for these rocking walls.

Introduction

Precast concrete rocking walls, which are designed with unbonded post-tensioning (PT) as the primary reinforcement across the connection interface between the wall and foundation, were developed as a potential design solution to overcome large residual drifts and significant damage to the plastic hinge regions that are associated with conventional reinforced concrete walls subjected to severe ground motions. Application of rocking walls in buildings is originated from the concept of seismic resilient systems, which were investigated in the PREcast Seismic Structural Systems (PRESSS) program [1]. The behavior of such systems differs from that of conventional cast-in-place (CIP) walls. The formation of plastic hinges in CIP walls enable significant energy dissipation through yielding of wall longitudinal reinforcement in the plastic

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hinge region adjacent to the wall base. Under lateral loading, the precast walls designed with unbonded post-tensioning undergo rocking motions with a large concentrated flexural crack opening at the wall to foundation interface. Since no energy dissipating element is included in this system, its energy dissipating ability is generally considered to be minimal as found from quasi-static testing. When a single rocking wall is subjected to seismic loading in real time, the rocking motion of the wall generates impact on the foundation, causing energy dissipation in the form of radiation damping. Hence, it will be of interest to investigate if this form of energy dissipation may be sufficient as the main source of damping in single rocking walls along with the inherent viscous damping of the wall.

Recently, several researchers, including Perez et al. [2], Kurama [3], Restrepo and Rahman [4], and Aaleti and Sritharan [5], presented different concepts and methods for designing and characterizing rocking precast concrete walls. While a significant progress on the design of these systems has been made, a systematic approach to understanding their dynamic response has not received sufficient attention. This paper presents an investigation of four single rocking walls (SRW) behavior subjected to a suite of input ground motions using a shake table. The influence of several design parameters such as initial prestressing, post-tensioning steel area and the confinement details in the wall corners near the base, on the seismic behavior of wall specimens were investigated.

Experimental Program

The wall test units identified as SRW1 through SRW4 were 6.25 ft long, 16 ft tall and 5 in. thick, and represented a 5/18 scale of a 6-story prototype office building in Los Angeles, California. The walls were designed using a concrete compressive strength, f'_c, of 6 ksi and ASTM A706 Grade 60 reinforcing steel. The measured concrete compressive strength of 8.5 ksi and 11.2 ksi achieved for SRW1-2 and SRW3-4 correspondingly.

Specimens were designed to be capacity protected against shear failure by providing adequate shear reinforcement to ACI 318-11 (2011) [6]. The confinement reinforcement in boundary elements in the wall corners of all the test units, were designed to sustain large compressive strains following the ACI ITG-5.2 guidelines [7]. As presented in Fig. 1, section A-A shows details of confinement reinforcement at the wall boundaries and section B-B shows 2 layers of No.3 shear stirrups with spacing of 15 in. along the entire height of the wall. As mentioned earlier, post-tensioning tendons (PT) play an important role in providing adequate restoring force to recenter the rocking wall, and therefore specific provisions should be considered in designing these unbonded PTs elongation during wall rocking. Although different amounts of PT areas and initial post-tensioning stress were considered for these walls to provide different lateral force resistance (as shown in Table 1), they were all designed to yield between 2 to 2.5% of the design-level lateral drift in compliance with ITG 5.2 guidelines [7]. Besides, strong impact of the rocking body on the foundation during severe ground motions may result in damage to confined concrete at the wall toes. To lessen the amount of damage, different configurations of wall base armoring were used for these four rocking walls; no armoring was used in SRW3, armoring of the entire wall length was done in SRW2 while only the wall toe regions were armored in SRW1 and SRW4 (see more details in Table 1). A steel fiber reinforced grout with a specified strength of 10 ksi was used at the interface between the wall base and foundation as suggested by ACI ITG-5.1 (2007) [8] and ACI ITG-5.2 [7].

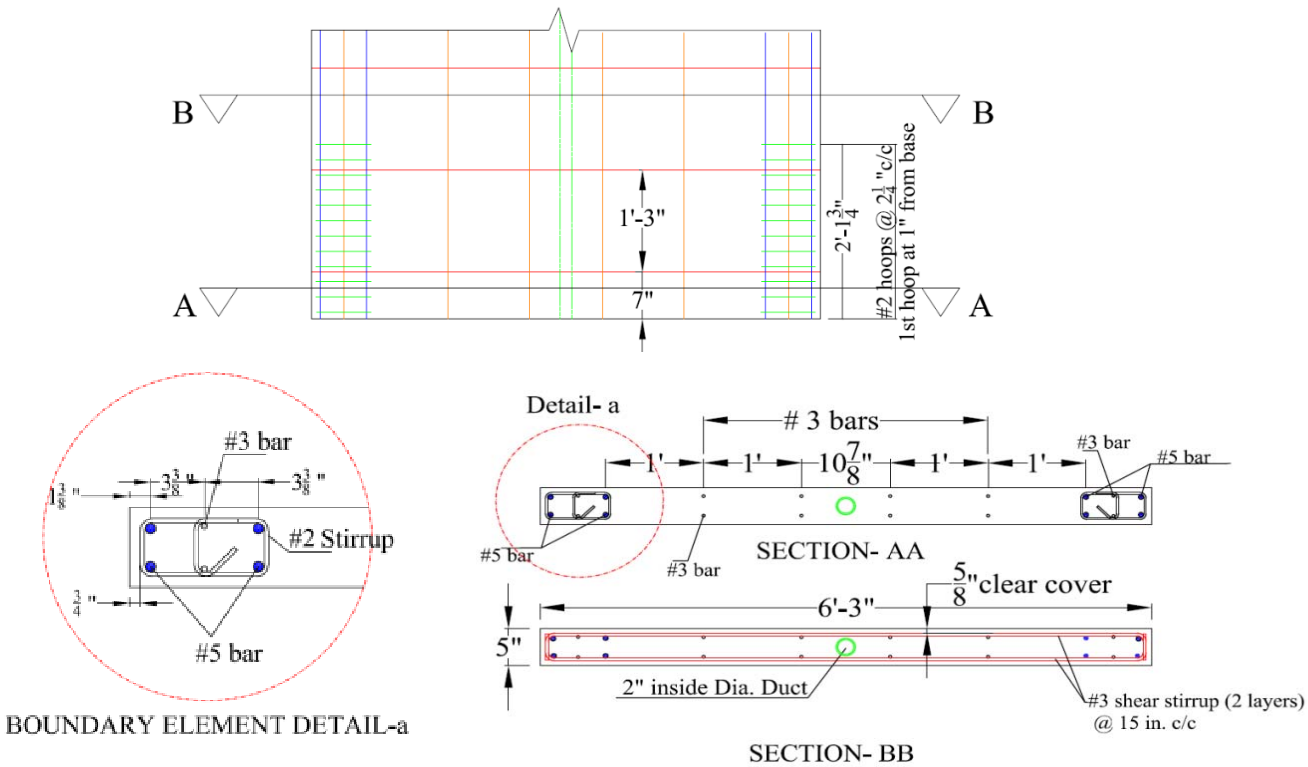


Figure 1. Typical cross-sectional details of all four walls (SRW1-4).

Design characteristics for 4 specimens are summarized in Table 1. Seismic height in this table indicates the height of seismic mass from base of the wall. As mentioned in the table footnote, all design expectations have not been achieved in experimental shake table testing.

Table 1. Test Variables for all specimens.

Specimen	Strand	Initial PT Stress (ksi)		Seismic Height (ft)	Base Channel Configuration
		Design	Measured		
SRW1	4,0.5"	172.00	170.00*	14	12" @ Both Corners
SRW2	6,0.5"	172.00	148.00	14	Along the wall
SRW3	6,0.6"	172.00	170.00	14	No Channel
SRW4	6,0.6"	172.00	166.00	11.5	15" @ Both Corners

*After Test-4, PT stress dropped to 79 ksi, due to large drift experienced due to resonance.

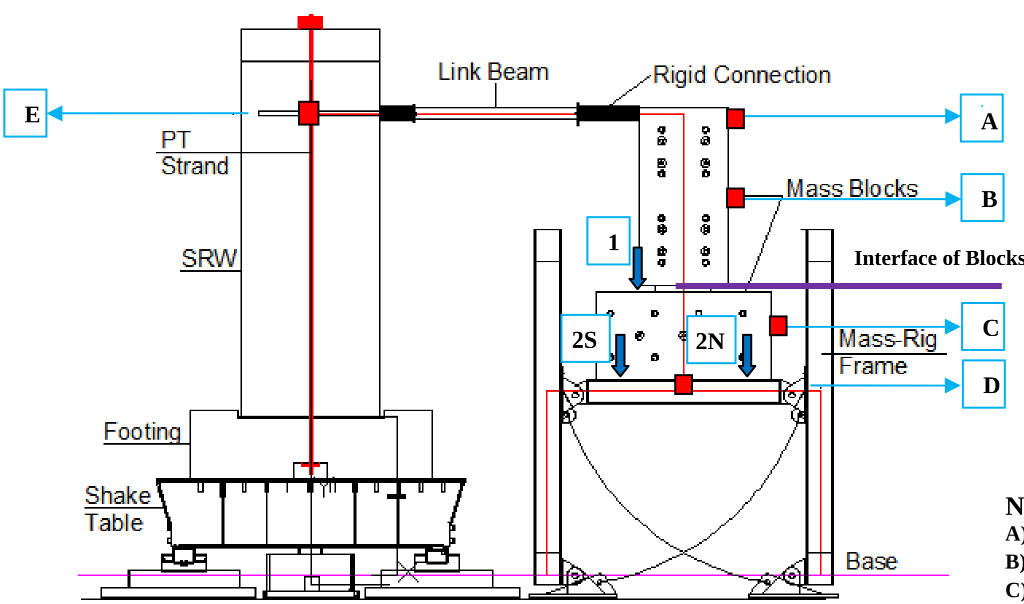
All specimens were subjected to a series of in-plane base excitations using the shake table set up at UNR, as shown in Fig. 2. The loading protocol for the tests included two suits of earthquake ground motions with varying intensities, sinusoidal harmonic excitations and free vibration testing. Only the wall responses to earthquake motions are presented here, which included two suites of records. The first suite consisted of short duration spectrum compatible motions as used for the pseudo dynamic testing of the PRESSS building [1] while the second suite consisted of recorded earthquake ground motions with appropriate scales as used by Rahman and Sritharan [9]. For the first suite of motions, the recommendations of SEAOC (2003) were used as the target spectra. Since the testing was done at 1/3.6 scale, the amplitude of all the motions was scaled up by 3.6 while the duration of the record was reduced by a factor 3.6.

Dynamic Assessment of the Experimental Set up

Simplifying the 6 story prototype building with an equivalent SDOF system and scaling it down by the factor of 3.6 led to applying a 60 kips seismic mass at 14' height of the wall from the base to ensure inertia effects would be accurately modeled. As shown in Fig. 2, a mass-rig system was used for applying the inertial load on the test wall secured to the top of the shake table. The mass rig frame itself provided 20 kips of the 60 kip, requiring an additional 40 kips that was provided using two concrete blocks (Fig. 2). A pin-ended rigid link beam was used to transfer the inertial load from the mass rig to the rocking wall.

Given the setup used for the test walls, it is needed to estimate base shear from governing dynamic equations of motion considering all forces generated within the system. This has been achieved through interpreting experimental data recorded by selected devices installed on mass-rig system and link beam as shown in Fig. 2. Experimental data indicates that total inertia force calculated from multiplication of each mass individually (20 kips of top and bottom block and also mass-rig frame) by the acceleration generated at the level of its center, equals to link beam force recorded by load cell. This result is presented in Fig. 3 for SRW3 excited by Kobe Earthquake. Accordingly, the base shear of the system is established as Eq. 1. In this equation, link beam force (load cell data) represents inertia force supposed to act upon mass-rig system and inertia force at Level E is generated by half weight of the wall, as effective seismic mass.

$$\text{Base Shear} = \text{Link Beam Force} + \text{Inertia force @ level E} \tag{1}$$



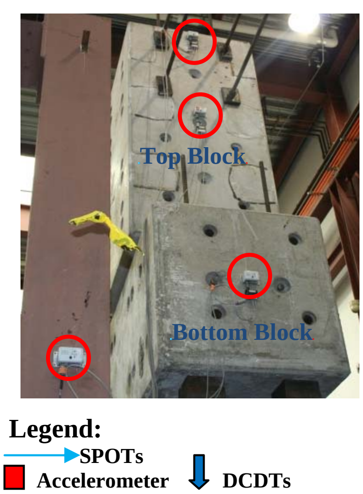


Figure 2. Instrumentation plan to study mass-rig system behavior.

Name of Devices at Specified locations:
A) Mass; Link beam level (Mass_LB)
B) Center of Top Concrete Block (Mass_CT)
C) Center of Bottom Concrete Block (Mass_CB)
D) Mass Rig Frame (MRF)
E) Wall; link beam level (Wall)
1) Between two concrete blocks
2) Between Bottom block and Mass Rig Frame

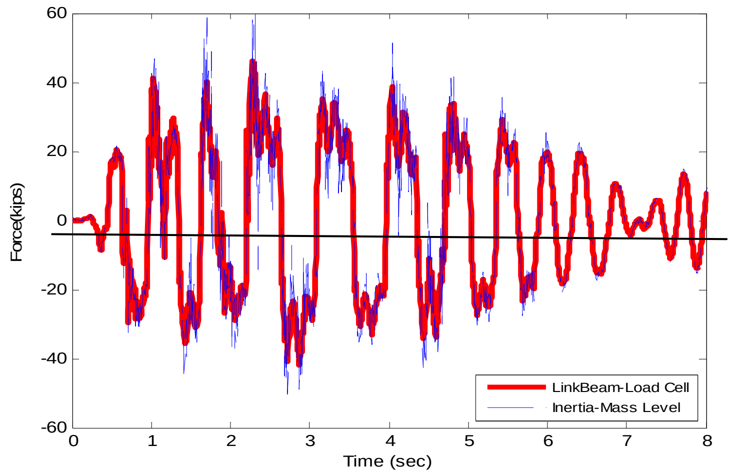


Figure 3. Inertia and link beam Force (Experimental Data) - SRW3-Kobe EQ.

Experimental Observations and Results

This section presents the critical observations from the overall performance of four single rocking walls specimens tested dynamically along with the comparison of their local and global behavior with simplified analysis. The results include global lateral force vs. displacement response of the wall system, post-tensioning stress variation, wall uplifts, wall panel contact length variation with lateral drift, and strain gauge responses in wall corners. Besides, impact of intensity and duration of ground motions on the maximum wall responses investigated.

Test Observations

Figs. 4a and 4b present examples of negligible damage observed at base as seen for SRW4 after it experienced a maximum lateral drift of 4.75% drift during the Kobe earthquake. While the use of the base channel helped reduce the damage, slightly more damage was seen when the wall base was not protected as seen in Fig. 4c. This damage occurred when SRW3 experienced a lateral drift of 3.1%.

a) Negligible Damage-Wall Base b) Negligible Damage-Wall Base c) SRW3-Without Base Channel

Figure 4. Experimental observations of shake table testing of rocking walls

Experimental Results

To understand the global characteristics of rocking walls, influence of different design parameters, such as the initial prestressing force on the behavior of specimens is first investigated in this section and experimental results are subsequently compared with results from a simplified analysis method. Then the impact of duration and intensity of ground motions on the maximum wall demands such as the drift, concrete strain and acceleration are studied.

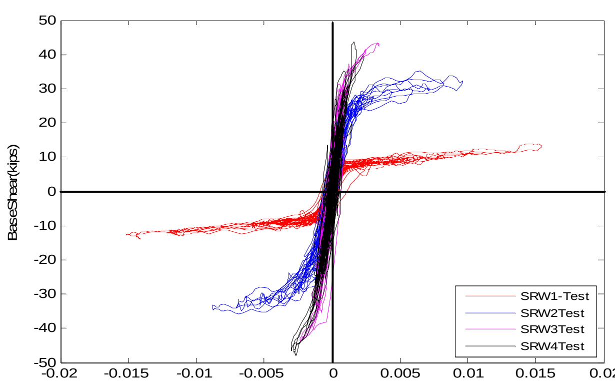
The lateral load responses of all the four specimens are presented in Fig. 5a by plotting the base shear obtained from Eq.1 as a function of the lateral drift while specimens subjected to the Chile EQ. As shown in this figure, the lateral resisting capacity of the rocking specimens improved with higher initial PT stress and increasing PT area. The measured initial PT force in SRW1-4 was 50.67, 137.6, 223.6 and 218.4 kips, respectively. It can be observed in Fig.5.a that SRW3 and SRW4 had identical behavior due to the same initial PT force. SRW1 had the lowest ultimate shear capacity and initial stiffness resulted from having the lowest initial PT force compared to other specimen. Also, as expected, the lateral load response of SRW2 lies between the SRW1 and SRW3 response.

In addition, lateral response of the rocking specimens was investigated for a case that resulted in PTs yielding due to larger amount of wall drift. For this purpose, the base-shear vs. drift of SRW2 was compared for the Kobe earthquake motion with amplification factors of 0.6 and 1.0, respectively (Fig. 5b). The expected lateral load response of SRW2 was calculated from a simplified analysis (SA) method proposed by Aaleti and Sritharan [5] also included in this figure. It should be noted that the initial and final prestressing force of SRW2 during the 0.6 Kobe EQ and 1.0Kobe EQ motions were recorded as (134.8, 119.2) kips and (119.2, 112.8) kips, respectively. PT strands yielded during testing to 0.6Kobe EQ, with the wall experiencing a drift beyond 2%. This caused reduction in PT force at the end of the motion. Bilinear idealization of experimental graphs, implies that yielding of PTs at the end of first test (0.6 Kobe EQ) decreased the initial stiffness of SRW2 during the following test (1.0 Kobe EQ). Consequently, wall started to rock and uplift at smaller drift with lower shear capacity. This reduction in stiffness resulted in increased drift and lead to concrete damage at wall toes during the stronger motion, which was more intense compared to a design level seismic event. As shown in Fig. 5.b, the simplified analysis method (SA) adequately predicted the lateral behavior of SRW2.

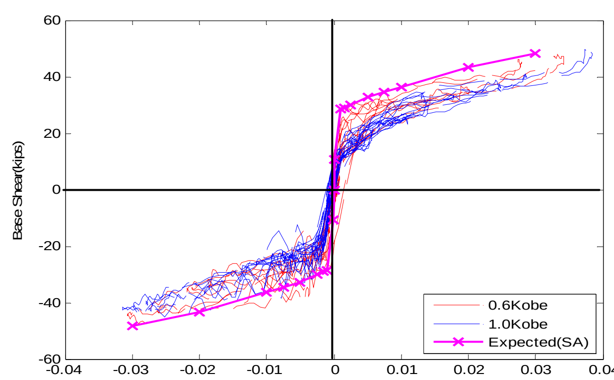
Fig.5.c shows the variation in the post-tensioning tendon forces in SRW1 as a function of peak lateral drifts. Linear relationship of PT force with drift confirms no yielding of strands in this test unit. Fig.5.d presents reduction in PT force while SRW3 endured a maximum of 2.6% drift during the application of 0.6Takatori motion. In this plot beginning and end of the test marked with green and red stars respectively. There was no residual drift at the end of testing, confirming the resilient performance of the system even after the PTs yielding.

As wall rocks from one corner to the opposite corner during motion, the compression toe region is defined within a specific distance from edge of the wall, called contact length. This impresses a region that concrete is more probable to experience large compressive strains and therefore should be confined with special reinforcement boundary elements. To estimate this length, maximum wall uplift measured by vertical DCDTs plotted along the length of the wall, while specimens experienced peak drift in both directions during motion. In Fig.5.e this process shown for SRW4, while the specimen excited by 0.6 Kobe earthquake and experienced maximum of 2.28% northward lateral drift and 1.1% lateral drift in the opposite direction (shown by star and diamond markers). Looking at the spread of data, a linear regression analysis performed to find the best fit straight line (R2 in Fig.5.e indicates R-squared or coefficient of determination for this analysis). Neutral axis length (contact length) then evaluated based on linear profile of the uplift along the wall length (noted as 9.669” & 5.924” in Fig.5.e). This process repeated for all motions excited the specimen and then, for different levels of first peak drifts, estimated contact length plotted and compared with trilinear idealization (SA method). As presented in Fig.5.f, for SRW4, experimental data points are close to the expected trend; however SA prediction results in smaller contact lengths. This may happened due to experiencing more damage at wall toes during shake table testing, which caused shifting the compression toe region more toward the wall center and therefore resulted in larger contact length, as observed in Fig.5.f.

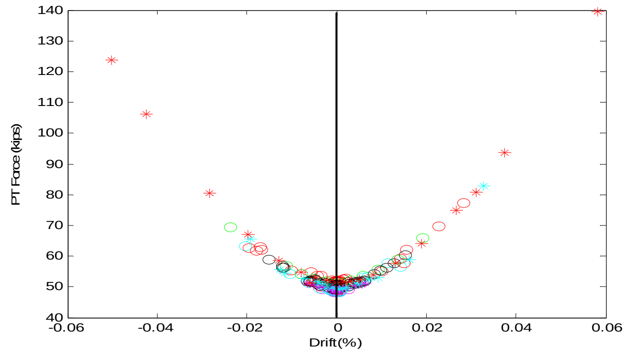
To come up with appropriate height of the confinement in the compression toe region, three gauges installed 4.5”, 12.5” and 20.75” from base of the wall at two end boundary elements. In Fig.5.g maximum recorded concrete compressive strain plotted as a function of device location for SRW2 for all shake table tests. Furthermore, to study the effect of increasing lateral drift in a given wall on maximum concrete strain, the most critical values plotted against first peak lateral drifts for the same specimen (Fig.5.h). As presented in these figures, maximum concrete compressive strain of 3372µ observed at 4.5” distance from base of the wall. However, Fig.5.h implies that for drifts less than 2.5% (marked with a red line), concrete strain demand in the confined regions was smaller than 0.003 in./in. Looking at Fig.5.g, also we can observe that moving away from base of the wall, confined concrete seems to experience no damage even for excitations stronger than design level seismic events. These experimental results indicate that height of provided confinement reinforcement at boundary elements can be reduced to wall thickness (5” for this specimen) for design purposes as recommended by ACI ITG-5.1 [8]. Furthermore, Fig.5.h provides a comparison between the critical experimental concrete strains and that estimated using an expression proposed by Aaleti and Sritharan [10]. Although a simplified method is used in this approach, expected results present a good estimate of experimental observations.



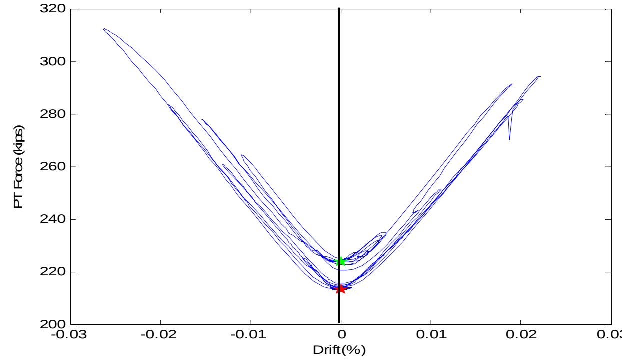
a) Lateral Response of SRW1-4 (Chile EQ.)



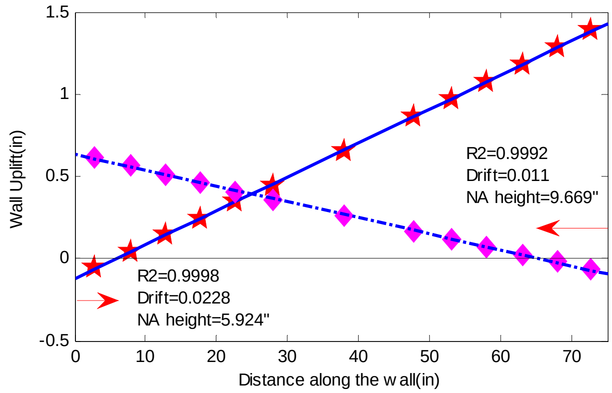
b) Lateral Response of SRW2 (0.6, 1.0 Kobe EQ.)



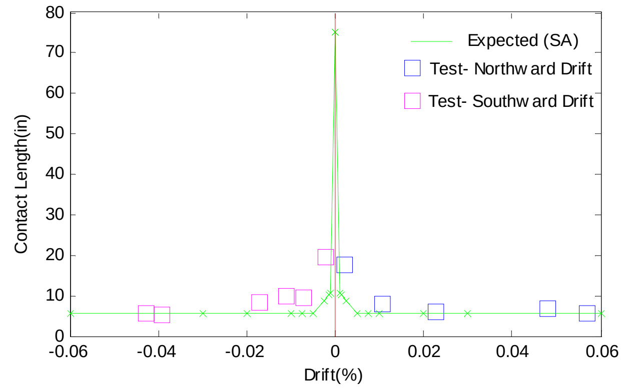
c) PT force vs. peak lateral drift of SRW1 (all motions)



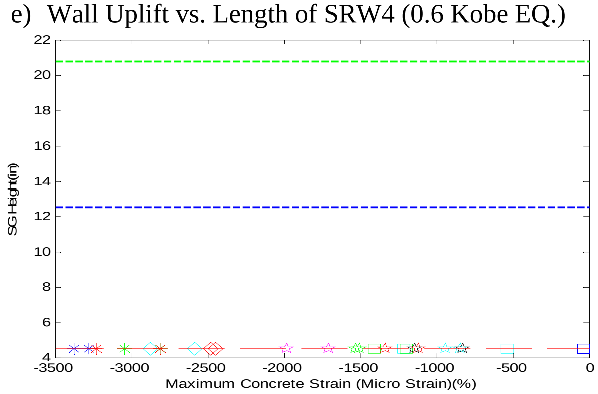
d) PT force vs. lateral drift of SRW3(0.6Kobe)



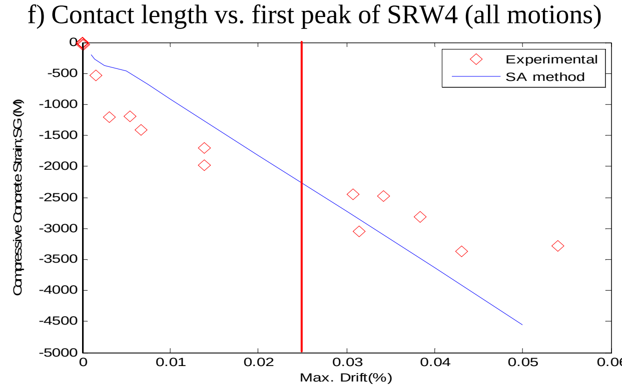
e) Wall Uplift vs. Length of SRW4 (0.6 Kobe EQ.)



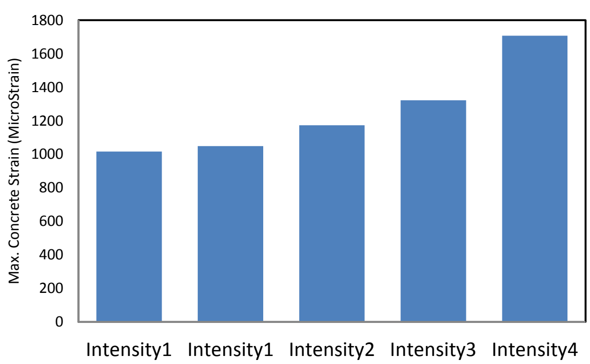
f) Contact length vs. first peak of SRW4 (all motions)



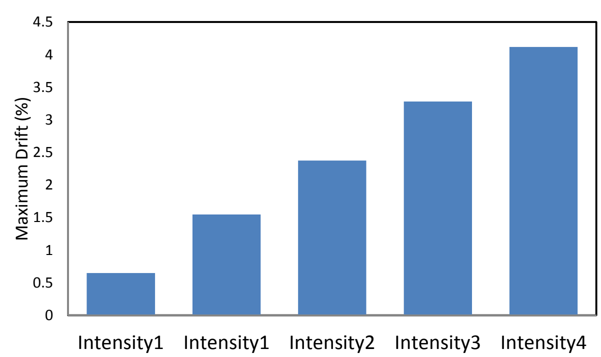
g) Distribution of concrete strain along height of SRW2



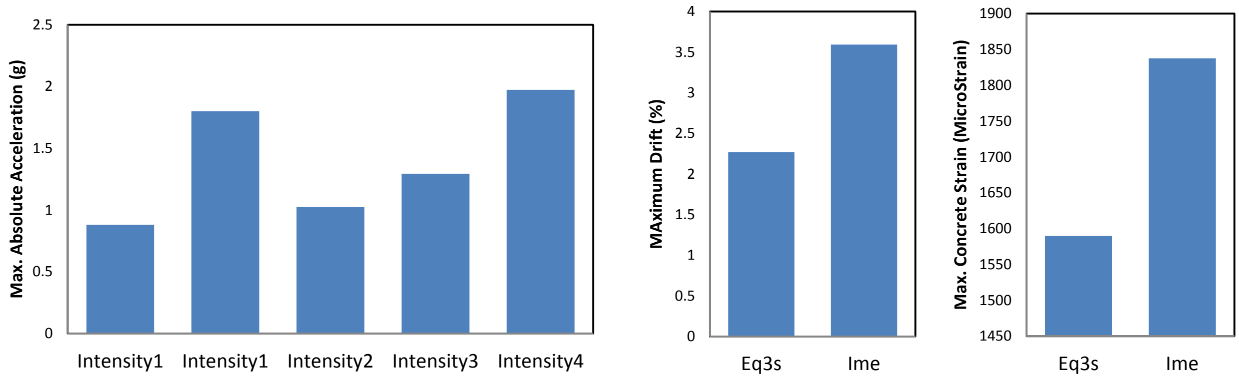
h) Wall Drift vs. max concrete strain of SRW2



i) Demand vs. motion intensity- Max Strain (SRW1)



j) Demand vs. motion intensity- Max Drift (SRW1)



k) Demand vs. motion intensity- Max Acc. (SRW1)

l) Demand vs. motion duration-Intensity3 (SRW1)

Figure 5. Global and Local Behavior of Specimens- Experimental Results

As mentioned earlier, an attempt to evaluate the influence of duration and intensity of ground motions on the behavior of rocking walls presented in this section. Figs.5.i-k show experimental response of SRW1 in terms of maximum wall demands applying second suite of ground motions described earlier in this paper. These results compared for different levels of earthquake intensities: New Zealand (Intensity 1), Chile (Intensity 1), Sylmar (Intensity 2), and Kobe-Japan (Intensities 3 and 4 applying two different amplification factors) respectively. As described before, different amplitude scaling factors applied to these earthquake records to comply with the Similitude. As presented in these figures, comparing wall responses to ground motions with different intensities, maximum drift and concrete strain increased with increasing the intensity. However, maximum absolute acceleration at top of the wall is larger for a motion with higher peak response spectrum, such as Chile and New Zealand record compared to Sylmar earthquake. Fig.5.l presents effect of duration of ground motions on the rocking wall response, comparing performance of SRW1 (in terms of maximum lateral drift and concrete strain) excited by a short duration spectrum compatible motion (Eq-3s) and similar intensity motion applied after that (Ime) identified as the first set of earthquake records in the paper. We can observe that motions with longer duration resulted in larger responses. Despite larger wall responses applying higher intensities and duration of excitations, it can be concluded that their overall behavior was satisfactory even for the maximum predictable design ground motion.

Conclusions

In this paper, results from shake table testing of four single rocking wall specimens are presented. According to the experimental observations, negligible damage was generally observed at base of the rocking walls, when they were subjected to design level earthquake ground motions. For more severe excitations, armoring the wall base using steel channels minimized the amount of damage. The lateral load behavior of the walls, estimated from Eq. 1, improved with higher initial prestressing stress and PT strand area. Also, PTs responded linearly until the design level drift was achieved. The post-tensioning steel experienced yielding at the higher drift; however, the resilient behavior of the wall system was not affected. The confinement detailing provided in boundary elements limited the maximum compressive strain of concrete to 0.003 for design-level earthquake loading, and therefore no critical damage to the confined concrete was observed. The simplified analysis (SA) method adequately predicted the experimental observations such as base shear vs. displacement of the system, length of the compression toe region and critical concrete strains in wall toe regions. As presented in this paper, although ground motions with higher intensities and durations resulted in larger maximum wall demands, overall performance of rocking specimens were acceptable even for the maximum credible earthquake record suggested by SEAOC (2003) for design of structures.

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