EVALUATION OF SEISMIC FAILURE MODES OF QUASI-BRITTLE REINFORCED CONCRETE SQUAT SHEAR WALLS THROUGH HYBRID SIMULATION AND QUASI-STATIC TESTS

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ABSTRACT

Squat reinforced concrete shear walls in nuclear structures tend to be very stiff and fail in a quasi-brittle manner characterized by a small deformation ductility capacity and a sudden drop in post-peak strength when transitioning to the sliding response mechanism. Current design code methods fail to adequately predict the strength and deformation capacity of such walls (Gulec et al. 2008). In order to better understand the failure modes and determine the strength and displacement capacities of these walls, a series of quasi-static and hybrid simulation seismic response tests were conducted.

Three nominally identical squat walls were tested. Two walls were tested at University of California, Berkeley with hybrid simulation using different sequences of ground motion inputs. One wall was tested with quasi-static cycles at University at Buffalo. The observed failure mode sequence and force-drift behavior were very consistent among the three tested walls, indicating that the sequence of ground motions did not have a significant impact on the global response, and also that the quasi-static test data is adequate to capture the global behavior of the walls. The peak shear strength of the tested walls was compared to predicted peak shear strengths from design code equations. The ASCE 41 backbone curve for modeling squat reinforced concrete shear walls, based on research by Wallace (2006), was evaluated and modified using the data from these wall tests.

INTRODUCTION

Low-aspect-ratio (squat) reinforced concrete walls often comprise the lateral-force-resisting system in nuclear and industrial structures. The design objective for these walls in nuclear structures is to maintain an elastic response during design basis earthquake (DBE) motions and to maintain an "essentially elastic" response during beyond design basis earthquake (BDBE) events. This is difficult to achieve because nuclear facility structures are very stiff and have short natural vibration periods, placing them into the acceleration sensitive range of a typical earthquake response spectrum, where even minor yielding results in a large increase in displacement ductility demand (Chopra 2007). Squat walls experiencing quasi-brittle failure modes lose strength and stiffness rapidly after the onset of inelastic deformation, so their displacement ductility capacities are very limited. In order to correctly determine the "essentially elastic" design target, the strength and displacement ductility capacities of the squat walls must be quantified.

Though design codes promote energy dissipation through the ductile mechanism of flexural yielding, squat walls tend to fail in shear and/or in sliding shear because their geometry does not allow them to bend easily. There is considerable uncertainty associated with the ability of current methods to predict the strength and deformation capacity of such walls (Gulec et al. 2008). Knowledge about the performance of these walls comes from test data, but stiff and brittle wall behavior and laboratory testing constraints both contribute to substantial scatter in the test data. Size effects associated with small-scale laboratory models and use of quasi-static loading sequences contribute to uncertainties about squat wall strength and displacement ductility capacity under ground motion excitations.

A series of quasi-static and hybrid seismic response tests were conducted to reduce these uncertainties. Three identical 20.3 cm (8 in) thick shear wall specimens modeled a prototype 91.4 cm (36 in) thick structural wall. The specimens had an aspect ratio of 0.53 and longitudinal and transverse reinforcement ratios of 0.67%. Two walls were tested using hybrid simulation of their seismic response at the University of California, Berkeley (Whyte and Stojadinovic 2012). The first wall experienced increasing levels of a ground motion, and the second wall experienced an initial large motion followed by two aftershocks. The third wall was tested at the University at Buffalo using a standard quasi-static cyclic incrementally increasing displacement test sequence (Rocks et al. 2011). The failure modes and the global force-drift behavior for the three walls are evaluated. The adequacy of design code equations for predicting the peak strengths of these walls is discussed first. Then, the ASCE 41 backbone curve for modeling squat reinforced concrete shear walls, based on research by Wallace (2006), is evaluated and modified to fit the observed data from these wall tests. The backbone curve given in the *Update to ASCE/SEI 41 Concrete Provisions* (Elwood et al. 2009) is shown in Figure 1. This curve shows the normalized shear capacity of the wall vs. the drift (wall top displacement/wall height). The strengths at points B and C are both taken to be the nominal shear strength ratio (V_n/V_n), which is calculated using Chapter 21.9 of ACI 318 (ACI 318 2008). Point F is the shear strength ratio at cracking (V_{cr}/V_n). Drift ratios are g=0.4%, d=1.0%, and e=2.0%. Strength ratios are c=0.2 and f=0.6.

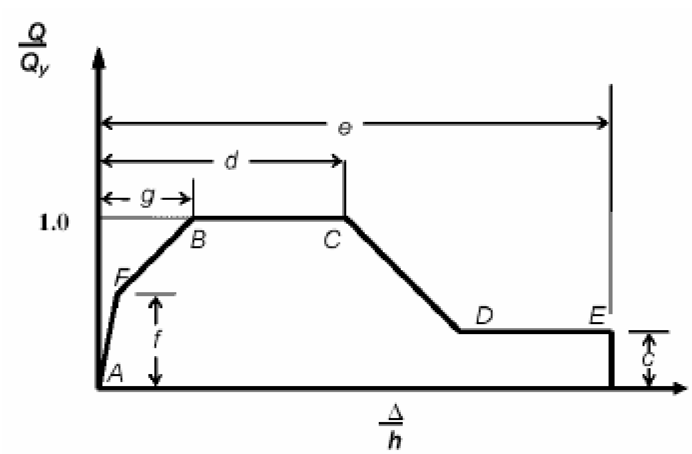


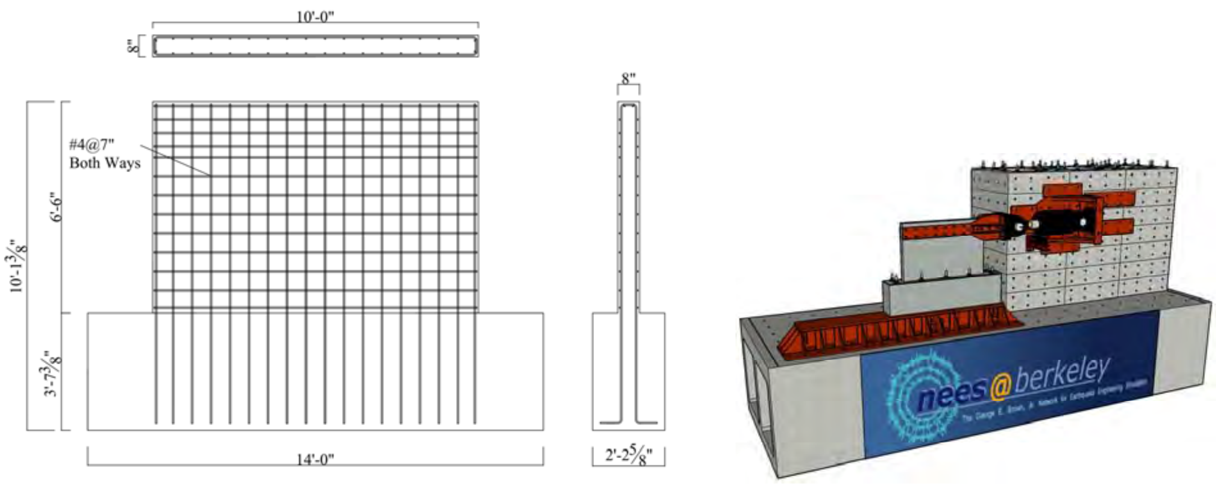
Figure 1. Load-displacement relationship for reinforced concrete squat shear walls (Wallace 2006).

HYBRID SIMULATION TESTS

Most previous squat shear wall experiments have employed either quasi-static cyclic or shaking table tests, usually using a small-scale specimen. The hybrid simulation method allows for testing a large-scale specimen using a realistically scaled ground motion excitation. More importantly, sequences of earthquake loads can be applied to the specimen. In hybrid simulation tests performed at the *nees@Berkeley* laboratory, the hybrid model consisted of a large-scale squat shear wall paired with a numerically modeled mass to achieve a natural period that was representative of a nuclear power plant structure. Due to the magnitude of the mass needed, it would not be possible to physically add this mass onto a shaking table or quasi-static test specimen. Two nominally identical walls were tested in hybrid simulation using two different earthquake sequences, one increasing in magnitude and the other simulating a large main shock followed by moderate aftershocks.

The shear wall specimens tested at the *nees@Berkeley* laboratory, referred to as Wall 1 and Wall 2, were 3 m (10 ft) long, 1.6 m (5 ft, 4-1/8 in) tall to the height of the actuator axis (aspect ratio 0.53), and

20.3 cm (8 in) thick. They had 0.67% horizontal and vertical wall reinforcement ratios with ASTM standard A706 #4 reinforcing bars placed in two curtains at 17.8 cm (7 in) on center. The yield stress of the reinforcing bars was 487.5 MPa (70.7 ksi). The concrete mix design had a target compressive strength of 34.5 MPa (5000 psi). The wall elevation and reinforcement layout is shown in Figure 2(a) and the global test setup is shown in Figure 2(b). The tall foundations were designed to accommodate the height of the actuator in the existing test setup at the *nees@Berkeley* laboratory. A single horizontal 6672 kN (1500 kip) +/-30.5 cm (+/-12 in) stroke hydraulic actuator was attached to a stiff loading arm, and uniformly spread the actuator load across the top of the specimen. No vertical load was applied to the wall specimen. Further details about the material properties and test setup are given in Whyte and Stojadinovic (2012).



(a) (b)

Figure 2. (a) Berkeley Wall elevation and reinforcement and (b) Test Setup at the *nees@berkeley* Equipment Site.

The numerically modeled mass was implemented using OpenSees (OpenSees 2013) and OpenFresco (OpenFresco 2013), so that the combined hybrid model matched the 0.14 s period of a Candu reactor prototype created by Yin-Nan Huang (Huang and Whittaker 2008, Huang et al. 2009). The 1999 Kocaeli, Turkey ground motion, matched to the Diablo Canyon Nuclear Power Plant DBE spectrum (Huang and Whittaker 2008), was used as the base motion for the hybrid simulations. It was further scaled to OBE (operational basis earthquake)-level (targeting 1/3 of the wall's yield force) and DBE-level (targeting 2/3 of the wall's yield force) expected behaviors of a nuclear facility structure. A BDBE motion was scaled to be an extremely large event that would enable investigation of the post-peak-strength behavior of the walls. It was selected to be 3 times larger than the DBE. The scaling factors, reported in Table 1, were applied to the Kocaeli base motion.

Table 1. 1999 Kocaeli ground motion scaling factors.

GM Level	Scaling Factor	PGA (g)
OBE	0.053	0.043
DBE	0.14	0.12
BDBE	0.42	0.35

The test sequences for Walls 1 and 2 are summarized in Table 2. Both walls initially experienced the OBE motion, since they were assumed to have been in service. Following the OBE motion, Wall 1 experienced a ground motion with a scaling factor of 0.11 (DBE 0.11), but the wall did not achieve the targeted 1/3 of its yield force for the DBE. Hence, the scaling factor of the DBE motion was increased to 0.14, as reported in Table 2, and this resulted in Wall 1 reaching the desired force. This 0.14 scaling factor was used for all remaining DBE and DBE aftershock motions. After the DBE, Wall 1 experienced the largest BDBE motion, followed by a DBE aftershock. Wall 2 experienced an OBE, then the BDBE, followed by two DBE aftershocks, to investigate how well the wall survived strong aftershocks after a very large event. After the ground motion excitations were completed, both walls were broken using cycles to 2.54 cm (1 in) and then 3.81 cm (1.5 in).

Table 2. Hybrid simulation ground motion sequences.

Wall 1	Wall 2
OBE	OBE
DBE 0.11	BDBE
DBE	DBE Aftershock 1
BDBE	DBE Aftershock 2
DBE Aftershock 1	

QUASI-STATIC CYCLIC TEST

A third nominally identical wall model was tested at the State University of New York at Buffalo using a quasi-static cyclic loading sequence at a loading rate of 0.102 mm/s (0.004 in/s) (Rocks et al. 2011). The Buffalo Wall had the following properties: aspect ratio of 0.54, 0.67% horizontal and vertical reinforcement, reinforcing bar yield stress of 434.4 MPa (63 ksi), and concrete compressive strength of 53.8 MPa (7800 psi). Figure 3 shows the test setup. No vertical load was applied to the wall.

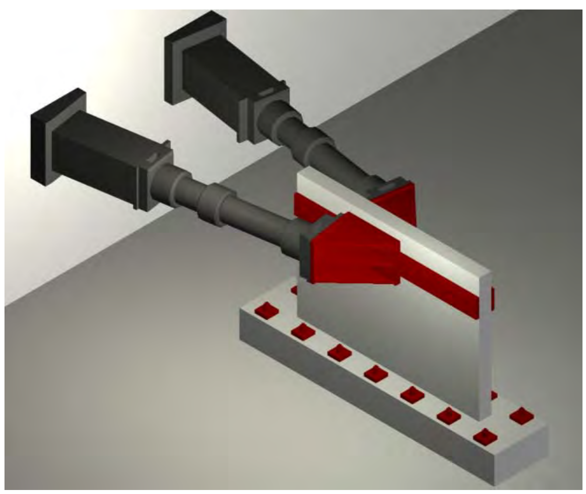


Figure 3. Test setup at the *nees@buffalo* Equipment Site.

TEST RESULTS

The sequence of failure modes was similar for all three specimens. During the OBE simulations, the Berkeley walls developed minor diagonal shear-induced cracks at about 45 degrees, distributed throughout the wall web. This indicated that the loading arm distributed the forces well along the length of the wall. The Buffalo wall developed similar diagonal cracks during the first load cycles. The shear cracks grew during the Wall 1 DBE simulation and during the beginning of the Wall 1 and Wall 2 BDBE simulations. As the BDBE motions progressed, some flexural cracks developed near the lower corners of the wall, and the flexural response mode became more dominant. The cracks in the lower left and right corners of the wall propagated along the entire base of the wall until they joined together, and the wall began to slide along this fully cracked zone at its base. After the wall had slid to the point where the vertical reinforcement engaged in dowel action, the flexural cracks continued to develop. The Buffalo wall exhibited a similar failure pattern as it experienced larger displacement cycles. Application of vertical load to the specimen in the physical test setup would have restricted this flexural behavior following sliding, and would have provided a more realistic response.

The final cycles experienced by the Buffalo Wall were 5.08 cm (2 in) in load step 13. The largest cycles experienced by both of the Berkeley walls were to 3.81 cm (1.5 in), consistent with the Buffalo wall load step 12. In order to compare the cracking patterns, the photograph of the Buffalo Wall at the first peak of load step 12 is compared to Berkeley Wall 1 at the end of the BDBE ground motion in Figure 4. The cracks in the Berkeley Wall 1 picture were enhanced using edge detection for better visibility, so the pictures in Figure 4 serve only to compare the crack pattern, not the crack widths. Additionally, the walls had experienced different motions and different total displacements at this point of comparison. Nevertheless, the Buffalo and Berkeley walls show a strong similarity in the locations of the shear cracks (both spacing and angle), and in the locations of the flexural cracks and their severity, relative to the shear cracks. Furthermore, the crack patterns between Berkeley Wall 1 and Wall 2 were also very similar, so there is consistency between the three walls.


(a)

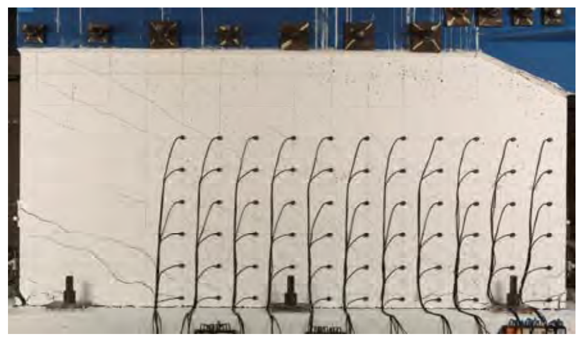

(b)

Figure 4. (a) Buffalo Wall at first peak of LS 12 and (b) Berkeley Wall 1 at end of BDBE motion.

Force-Deformation Behavior

The peak force developed by Wall 1 was 1633 kN (367.1 kips), Wall 2 was 1710 kN (384.4 kips), and the Buffalo wall was 2082 kN (468 kips). Figure 5 shows the shear strength normalized by the product of wall area and $\sqrt{f'_c}$ vs. drift response of the three walls. The Berkeley OBE motions are compared to Buffalo Load Steps 1-2. The DBE motion for Wall 1 (Wall 2 did not experience this motion) is compared to Load Steps 3-6. The Berkeley BDBE motions are compared to Load Steps 7-11. Finally, the DBE Aftershocks and cycles to 2.54 cm (1 in) and then 3.81 cm (1.5 in) are compared to Load Step 12.

The sudden drop in specimen resistance in the Berkeley wall plots when the direction of the actuator motion changes, which is most apparent in the BDBE motion, is due to slip in the actuator's clevises. The clevis gaps are closed when the actuator is pushing the specimen, but upon direction reversal, the clevis gaps must open before the specimen begins to move. As the clevis gaps are opening, the force on the wall is decreasing yet the wall itself is not yet moving. Additionally, the response of both Berkeley specimens to the selected ground motions was asymmetric. In order to confirm that this was the result of the nature of the ground motion and not an asymmetry in the test setup, the ground motion direction was reversed for the Wall 2 hybrid simulation by changing the algebraic sign of the acceleration array. To facilitate comparison between the walls, the first quadrant in the response plots corresponds to the actuator pulling Wall 1 and pushing Wall 2. The Buffalo Wall did not show this asymmetry. It achieved a slightly larger drift in the positive direction and a drift comparable to the Berkeley walls in the negative direction. The Buffalo Wall experienced higher peak strengths than the Berkeley walls, but it also had a larger concrete compressive strength, so its normalized shear strength was slightly lower than that of the Berkeley walls. The Buffalo Wall hysteresis showed more pinching than that of the Berkeley walls. This is likely a result of greater slip in the Berkeley actuator clevises. In general, the three walls showed very similar force-deformation behavior. The similarity between the observed sequence of cracking patterns and the force-deformation plots of Wall 1 and Wall 2 suggests that the sequence of the ground motions does not significantly affect their force-drift global response. The similarity between the Berkeley and Buffalo walls suggests that a quasi-static test is adequate to capture the global behavior of these quasi-brittle specimens. Local effects could not be compared for the three walls. In the Wall 1 test, the gain on the data acquisition system was set too high, so the strain gages were saturated at low strains. The Berkeley and Buffalo walls had different strain gage layouts, so this comparison did not yield useful information.

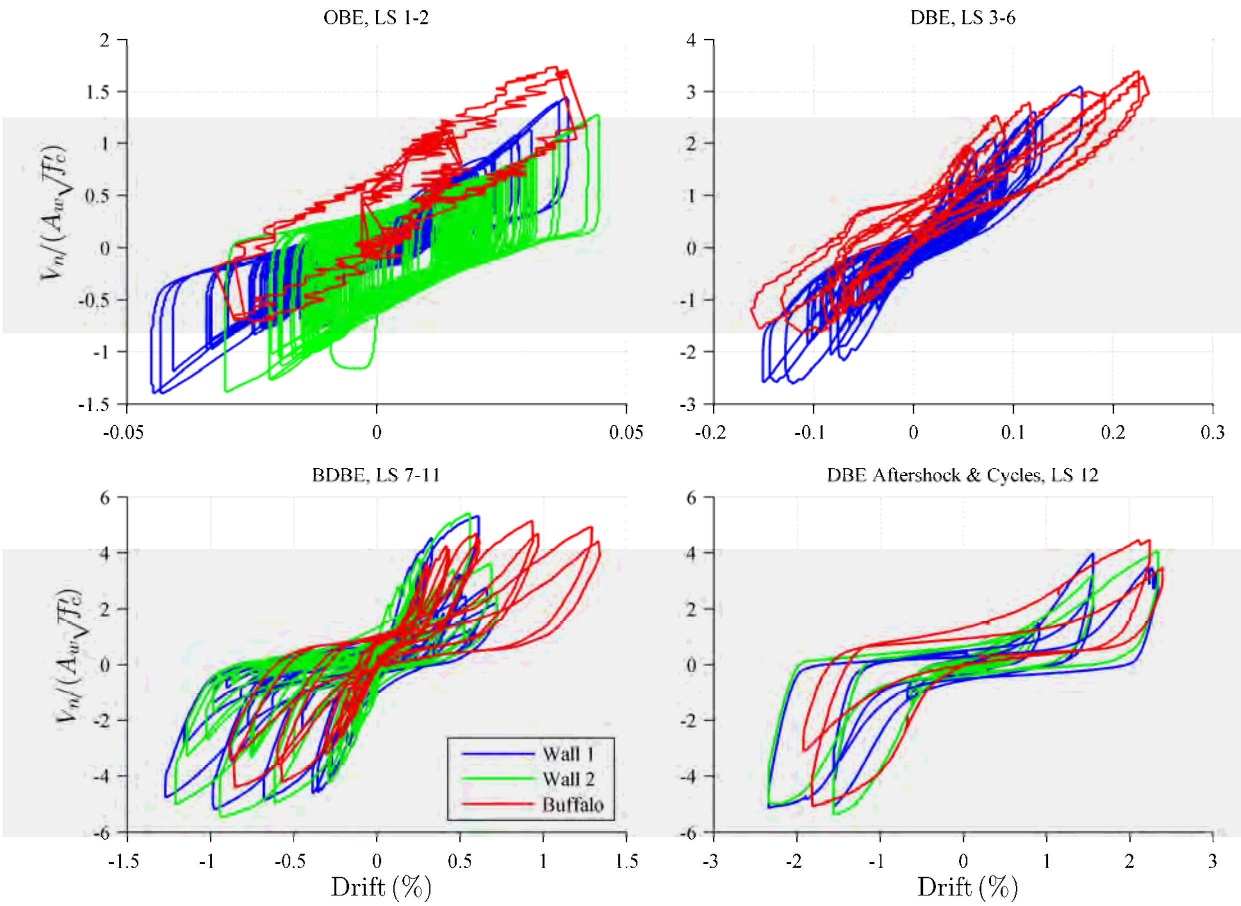


Figure 5. Normalized shear strength vs. drift response for Berkeley Walls 1 and 2 and the Buffalo wall.

Design Code Equations for Peak Shear Strength

Figure 6 shows the ratio of predicted to measured wall strength using design code equations from Gulec and Whittaker (2009), Wood (1990), ACI 318 Chapter 11.6, 11.9, and 21.9 (ACI 318 2008), ASCE 43 (ASCE 2007), and Barda et al. (1976). ACI 318 Chapter 11.6 (ACI 318 2008) provided the closest prediction of the strength of the Buffalo wall. Wood's (1990) equation best predicted the peak strength of the Berkeley walls. As reviewed previously, the Buffalo wall had a higher concrete compressive strength and thus achieved a larger peak force. In general, the code equations predicted the strength of the Buffalo wall better than the Berkeley walls, but most equations significantly overestimated the peak strength of the walls. The Barda (1976) equation over-predicted the Wall 1 peak strength by nearly a factor of 2. The ASCE 43, ACI Chapter 21.9, and Barda equations that significantly over-predicted the responses are more appropriate for predicting peak strengths of walls with barbells or flanges. For characterizing the behavior of walls using design equations, it will be important to make this distinction.

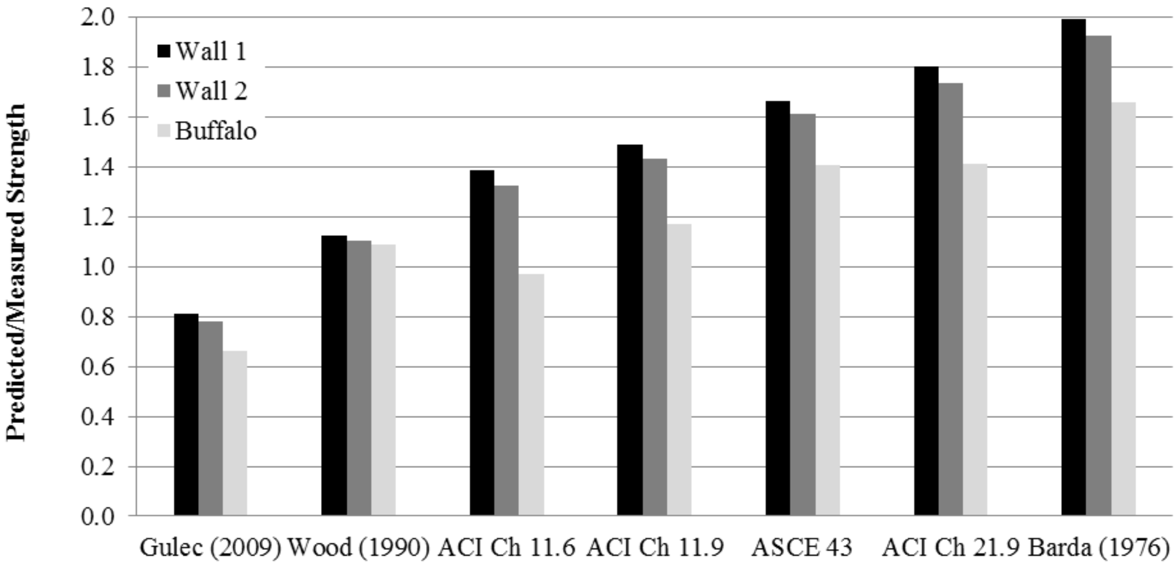


Figure 6. Ratio of predicted to measured strengths for various strength equations.

Modified Wallace Backbone Curve

A backbone curve enveloping the measured force-deformation response data presented in Figure 5 is constructed starting with the Wallace Envelope shown in Figure 1. In order to better fit the observed test data, the Wallace Envelope is modified in two ways. The Wallace Envelope uses the peak shear strength equation from ACI 318 Chapter 21 (ACI 318 2008), but this equation was shown to significantly overestimate the response observed in these squat wall tests. From Figure 6, Wood's (1990) equation (namely, $6\sqrt{f'_c}$, the lower bound of peak shear stress in squat shear walls) provides the closest upper-bound estimate of the peak shear strength for the three walls: Wall 1 peak shear stress was $5.32\sqrt{f'_c}$ psi; Wall 2 peak shear stress was $5.45\sqrt{f'_c}$ psi; and the Buffalo wall's peak shear stress was $5.52\sqrt{f'_c}$ psi. Thus, Wood's (1990) equation is used here to draw the Modified Wallace Envelope. Second, Wallace recommended using $0.4E_c$ to estimate the uncracked initial shear stiffness of a wall, which was much higher than observed in these tests. The initial stiffness estimate made using $0.12E_c$, 30% of the Wallace recommendation, is used to draw the Modified Wallace Envelope in Figure 7. With these two changes, the Modified Wallace Envelope closely approximates, up to the point of peak strength, the backbone curves for the three tested walls. Beyond peak strength, the measured wall response backbone curves are significantly stronger than the Modified Wallace Envelope. Since the walls tested in this study did not have any vertical load physically imposed on them in the laboratory, additional strength developed through longitudinal reinforcement re-engagement in bending following sliding that may not be realistic. Additional research based on more sophisticated finite element models of the squat walls to determine the post-peak force-deformation response envelope for squat walls is ongoing.

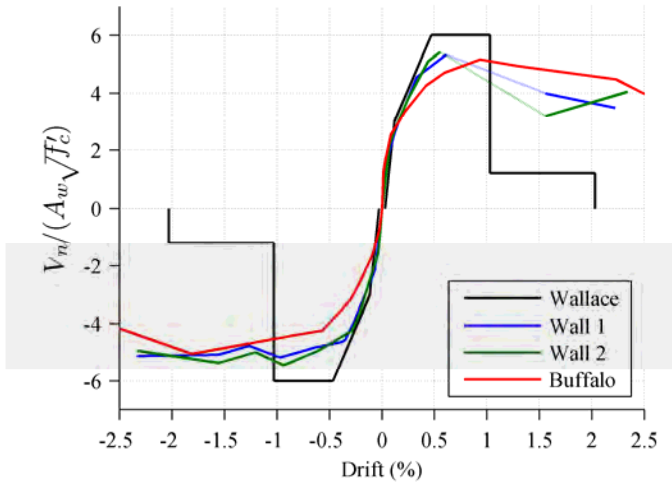


Figure 7. Modified Wallace Envelope compared to the experimentally obtained normalized shear strength vs. drift envelopes.

SUMMARY AND CONCLUSIONS

Two squat shear walls were tested using sequences of ground motions in hybrid simulation tests at the *nees@Berkeley* Laboratory. A nominally identical wall was tested at the State University of New York at Buffalo using a conventional quasi-static cyclic loading pattern. The global behavior was similar for the three wall specimens. This was demonstrated through similar development of cracking patterns: from shear behavior, to flexural response, and finally to sliding shear response. This was also shown in similar force-deformation plots for all three walls. These similarities indicate that the varying ground motion sequences of the Berkeley walls do not affect the global response, and furthermore, the quasi-static test of the Buffalo wall was adequate to capture the global wall behavior in seismic events. A comparison of local behavior was not possible in this study. The gain on the data acquisition system was set too high for the Berkeley Wall 1 test, so the strain gages were saturated at low strains. The Buffalo wall strain gages were not placed in the same locations as the Berkeley walls, so this comparison was not useful.

Various code equations were evaluated for their ability to predict the peak shear strength of the experimentally tested walls. Most equations overpredicted the strength; in the worst case, by a factor of almost 2. The wall strength prediction equation from above that came the closest to the experimentally observed strengths was the one proposed by Wood (1990). The equations that gave the most significant overestimates of peak strength were based on results from wall tests with barbells or flanges, so it is important to classify code equations based on different wall types. The rectangular walls tested in this study did not have any vertical load, a condition that also needs to be accounted for in the selection of an adequate equation for predicting the strength of squat shear walls.

Finally, the Wallace (2006) force-deformation response envelope was modified using $0.12E_c$ to estimate the initial stiffness of a squat wall and $6\sqrt{f'_c}$ upper-bound shear stress to estimate the peak shear force capacity of a squat wall. The Modified Wallace Envelope approximates the experimentally observed wall backbone force-deformation response envelopes well up to the peak force. Since the walls tested in this study did not have any vertical load physically imposed on them in the laboratory, they re-engaged the longitudinal reinforcement in flexure after significant sliding, a response mechanism that may not occur in the presence of significant vertical (gravity) loads. Therefore, the post-peak force-deformation response and the deformation capacity of the tested walls could not be accurately estimated using global force-deformation behavior models. Further work to determine the displacement ductility capacity of squat reinforced concrete shear walls that fail by shear and sliding is ongoing.

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