Proceedings of the International Symposium on Engineering Lessons Learned from
the 2011 Great East Japan Earthquake, March 1-4, 2012, Tokyo, Japan

BEHAVIOR AND DESIGN OF STRUCUTRAL WALLS – LESSONS FROM RECENT LABORATORY TESTS & EARTHQUAKES

John WALLACE[1] and Jack MOEHLE[2]

[1] Professor, Department of Civil Engineering, University of California, Los Angeles,
Los Angeles, CA, USA, wallacej@ucla.edu
[2] Professor, Department of Civil Engineering, University of California, Berkeley,
Berkeley, CA, USA, moehle@berkeley.edu

ABSTRACT: Design and construction practice for structural walls has evolved significantly over the last 20 years and engineers have pushed design limits in recent years, optimizing economy and design, and in many practices producing walls with higher demands and more slender profiles than have been verified in past laboratory testing or field experience. Wall performance in recent earthquakes and laboratory tests is reviewed and US code provisions are reassessed to identify possible shortcomings and improvements.

Key Words: 2010 Chile earthquake, structural wall, shear wall, testing, splice, ACI 318, detailing, displacement-based, slenderness, instability

INTRODUCTION

Design and construction practice for special structural walls (ACI 318 designation) has evolved significantly since the system was introduced in the 1970's. Throughout the 1970s and 1980s, it was common to use so-called barbell-shaped wall cross sections, where a “column” was used at each wall boundary to resist axial load and overturning along with a narrow wall web. In the late 1980s and early 1990s, use of rectangular wall cross sections became common to produce more economical designs. Use of walls with boundary columns is still common in Japan; however, based on information available in the literature, the AIJ Standard for “Structural Calculations of Reinforced Concrete Buildings” was revised in 2010 to show RC walls with rectangular cross-sections. Engineers around the world have pushed design limits in recent years, optimizing economy and design, and in many practices producing walls with higher demands and more slender profiles than have been verified in past laboratory testing or field experience. The trend towards more slender profiles has been accelerated by use of higher concrete strengths.

Observed wall damage in recent earthquakes in Chile (2010) and New Zealand (2011), where modern building codes exist, exceeded expectations. In these earthquakes, structural wall damage included boundary crushing, reinforcement fracture, and global wall buckling. Recent tests of isolated structural walls in the US and tests of two, full-scale 4-story buildings with high-ductility structural walls at E-Defense in December 2010 provide vital new data. A particularly noteworthy aspect of these recent tests is the failure of relatively thin wall boundaries to develop ductile behavior in compression, even though they complied with building code provisions and recommendations of ACI and AIJ.

The observed performance following recent earthquakes and in recent laboratory tests strongly suggest that the problems observed are not isolated and that analysis and design provisions need to be reassessed. In particular, the quantity and configuration of transverse reinforcement required at wall boundaries needs to be reassessed to address issues associated with wall thickness, slenderness, axial load, and configuration, as well as expected displacement demands and load history. Preliminary studies indicate that greater amounts of transverse reinforcement may be required for thin walls or walls with large cover and that tighter spacing of transverse reinforcement may be required to suppress buckling of vertical reinforcement, especially for walls with light axial load or walls with flanges. These findings apply to both high ductility (ACI Special) and low ductility (ACI Ordinary) walls.

OBSERVED PERFORMANCE OF STRUCTUAL WALLS

Recent Earthquake Reconnaissance

Recent earthquakes in Chile (M_w 8.8, February 2010), New Zealand (February 2011, M_L=6.3), and Japan (M_w 9.0, March 2011) have provided a wealth of new data on the performance of modern buildings that utilize structural walls for the primary lateral-force-resisting system. Although complete building collapse was rarely observed, damage was widespread and generally exceeded expectations.

In 1996, Chile adopted a new code (NCh 433.Of96, 1996) based on ACI 318-95 and produced an immense inventory of progressively more slender buildings corresponding essentially to the US reinforced concrete code provisions, except boundary element confinement was not required. The 2010 M_w 8.8 earthquake caused serious damage to many of these buildings, including crushing/spalling of concrete and buckling of vertical reinforcement, often over a large horizontal extent of the wall (Fig. 1). Damage tended to concentrate over a relatively short height of one to three times the wall thickness, apparently because buckling of vertical bars led to concentration of damage. Closer inspection of the wall boundary regions (Figure 1) revealed the relatively large spacing of hoops (20 cm) and horizontal web reinforcement (20 cm), as well as the 90-degree hooks used on hoops and horizontal web reinforcement, which may have opened due to concrete crushing and/or buckling of vertical reinforcement (Fig. 1d). Some of the failures are attributable to lack of closely-spaced transverse reinforcement at wall boundaries, which was not required by the Chilean code based on the good performance of buildings in the 1985 M7.8 earthquake; however, many of the failures are not yet understood, and many suggest that there are deficiencies in current US design provisions (Wallace, 2011; Massone and Wallace, 2011). In some cases, lateral instability (buckling) of a large portion of a wall section was observed (Fig. 2); prior to the Chile and New Zealand earthquakes, this global buckling failure had been primarily observed in laboratory tests (e.g., Thomsen and Wallace, 2004). Detailed surveys conducted as part of ATC-94 (2011) indicate that global wall buckling was not driven by prior yielding in tension (as had originally been suspected based on past research, e.g., Corley et al., 1981; Paulay and Priestley, 1993; Chai and Elayer, 1999) but instead was the result of lateral instability of previously crushed boundary zones. Furthermore, the ATCI-94 study has been unable to establish through analysis the role of pre-emptive longitudinal bar buckling as a trigger for compression failure of lightly confined boundary zones. Laboratory testing is required to understand these behaviors; preliminary studies are underway in Chile and the US to investigate these issues.

The 2011 Christchurch earthquake (EERI, 2011; NZRC, 2011) shows many similar wall failures, suggesting the deficiencies observed in the 2010 Chile earthquake are not isolated (Fig. 3a). All of the walls depicted in Fig. 2 and 3 have either T-shaped (Fig. 2, 3b) or L-shaped (Fig. 3a) cross sections, which leads to large cyclic tension and compressive demands at the wall web boundary (Wallace, 1996).

The wall web boundaries are apparently susceptible to out-of-plane buckling following cover concrete spalling. Although current ACI 318-11 provisions require consideration of an effective flange width, the provisions do not restrict use of narrow walls and do not address this out-of-plane failure mode, i.e., there are no restrictions on wall thickness or wall slenderness. Failure of diaphragm-to-wall connections were observed in Christchurch, potentially contributing to the collapse of the several buildings (Elwood, 2011). In Chile, typical buildings have a large number of walls that well-distributed in plane; therefore, diaphragm failures were not observed.

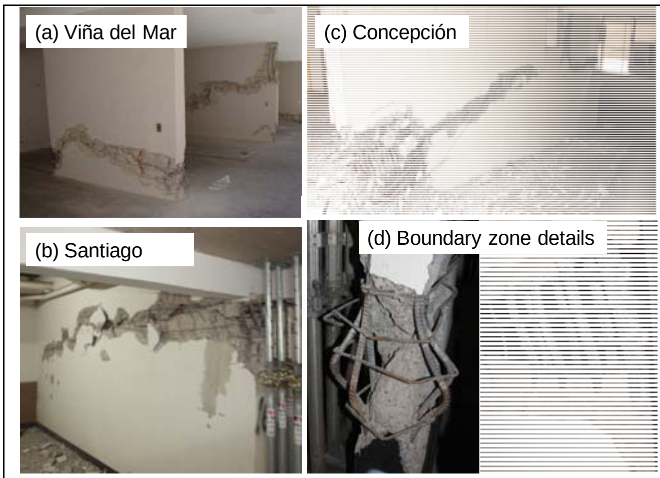


Figure 1 Typical wall damage in Chile Earthquake

Figure 2 Wall lateral instability


Figure 3a Wall failure in 2011 Christchurch earthquake (Elwood, 2011)


Fig. 3b Specimen TW2 web boundary failure (Thomsen and Wallace, 2004)

Recent laboratory studies of conventional walls

Recent laboratory testing of structural walls in the US has focused on addressing concerns related to behavior of walls with rectangular and T-shaped cross sections subjected to uniaxial and biaxial loading (Waugh et al., 2008; Waugh and Sritharan, 2010; Brueggen and French, 2010), with couplers and splices in the plastic hinge region (Johnson, 2010, Birely et al., 2010), with higher shear demands (Birely et al., 2008; Birely et al., 2010; Sriram and Sritharan, 2010), and with coupling beams (Naish and Wallace, 2010; Para-Montisenos et al, 2012; Lehman and Lowes, 2011). All of these studies involved quasi-static testing. Shake table testing of walls has been limited, except for 7-story “building slice” tests of walls with rectangular and T-shaped cross sections conducted by Panagiotou and Restrepo (2007). The overwhelming majority of quasi-static and shake table tests conducted in Japan have been conducted on barbell-shaped walls and low-rise buildings with “wing walls” (e.g., Kabeyasawa et al., 2008, 2010(a), 2010(b)), which are not common in the US. Only recently have the Japanese Building Standard Law and Architectural Institute of Japan (AIJ, 2010) recommendations been modified to allow the use of rectangular walls with boundary elements, but their use is not widespread.

Johnson (2010) reports test results isolated, slender (h_w/l_w and $Mu/V_u l_w = 2.67$) cantilever walls to investigate the behavior of anchorage details. Three walls were tests, one each with continuous (RWN), coupled (RWC), and spliced (RWS) vertical reinforcement. The wall cross sections were 6 in. x 90 in. (152.4 mm x 2.29m), and the walls were subjected to horizontal lateral load approximately 20ft or 6.1m above the base). Although the wall cross-sections were rectangular, different amounts of boundary vertical reinforcement were used to simulate the behavior of T-shaped wall cross sections; 4-#6 (d_b=19mm) and 2-#5 (d_b=15.9mm) at one boundary and 8-#9 (d_b=28.7mm) at the other boundary. Horizontal wall web reinforcement, of #3 @7.5 in. or $\rho_t = 0.0049$ (d_b=9.5mm @ 19cm), was selected to resist the shear associated with the expected moment strength (including overstrength). Wall web vertical reinforcement consisted of #4 @18 in. or $\rho_v = 0.0037$ (d_b=12.7mm @ 45.7cm). It is noted that the 18 in. (45.7cm) spacing of vertical web reinforcement is the maximum spacing allowed by ACI 318-11 21.9.2.1. It is questionable whether such a large spacing (45.7cm) in such a thin wall (15.2cm), satisfies the intent of R21.9.4, which states that wall we reinforcement should be "appropriately distributed along the length and height of the wall… should be uniform and at a small spacing." Because a construction joint exists at the foundation-wall interface, shear friction requirements of ACI 318-11 11.6.4 should be satisfied. Given that 4 pairs of #4 vertical bars are used, the shear friction strength V_n is 96k (427kN) and 58k (256kN) for a coefficient of friction $\mu = 1.0$ and 0.6, respectively (neglecting the boundary vertical reinforcement). Lateral load versus top lateral displacement relations for RWC and RWS are plotted in Fig. 4a; since results for RWC and RWN are very similar. For RWC, the wall reach rotations exceeding +0.035 (#5 in tension) and -0.02 (#9 in tension), whereas for RWS, the wall reach rotations of approximately +0.02 (#5 in tension) and -0.012 (#9 in tension). Damage was concentrated at a single, large crack at the foundation-wall interface, which accounted for about 0.015 rotation at a top rotation of 0.02. The crack width reached more than It is noted that the applied shear is close to or exceeds the shear friction capacity V_n of the walls, depending on the direction of the applied load and the value assumed for the coefficient of friction. Significant horizontal cracking also was observed for specimens RWN and RWC, suggesting that the quantity (and large spacing) vertical web reinforcement was insufficient to restrain sliding, and damage concentrated at the foundation-wall interface for specimen RWS (Fig. 4b). However, the test results do indicate that the presence of the splice significantly reduced the wall lateral deformation capacity.

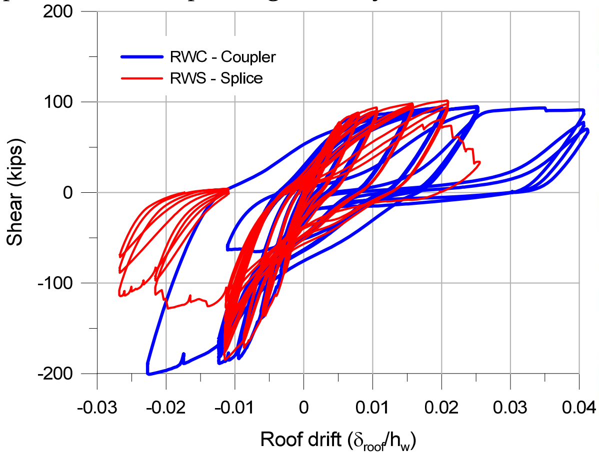


Fig. 4a Load-displacement relations

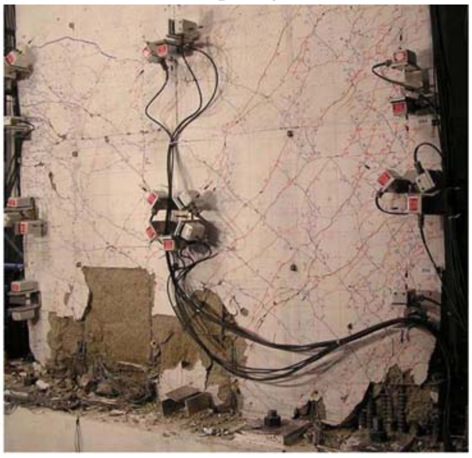


Fig. 4b Wall damage at end of test (RWS)

Tests of walls with splices also were conducted by Birely et al. (2010). The test specimens were roughly one-half scale replicas of the bottom three stories of a ten-story wall (Fig. 5a. Base shear versus 3rd story (top) displacement plots are shown in Fig. 5b for three of the tests, PW1 (splice, $M_b=0.71h_wV_b$), W2 (splice, $M_b=0.50h_wV_b$), and W4 (no splice, $M_b=0.50h_wV_b$). Design wall shear stresses were 0.23, 0.33, and 0.33$\sqrt{f_c^{'}\ MPa}\ MPa$ for W1, W2, and W4, respectively (equivalent to 0.7, 0.9, and 0.9V_n). The #4 (d_b=12.7mm) boundary bars were lapped 0.61m, with spacing of boundary transverse reinforcement of 51mm (s/d_b =4). The test with lower shear stress was reasonably ductile, achieving 1.08M_n and a 3rd story lateral drift of 1.5% prior to strength loss; however, test PW4, with no splice, reached only 1.0% lateral drift at the third story (top) prior to strength loss. For all tests with splices, damage initiated with buckling of the interior bar at the wall edge (Fig. 6a) and then concentrated at the top of the splices (Fig. 6b), whereas damage was concentrated at the foundation-wall interface for test PW4 with no splice (Fig. 6c). Even without consideration of the elastic deformations over the top seven stories not included in the test, deformation capacities of the walls are less than expected, especially for PW4, with no splice.

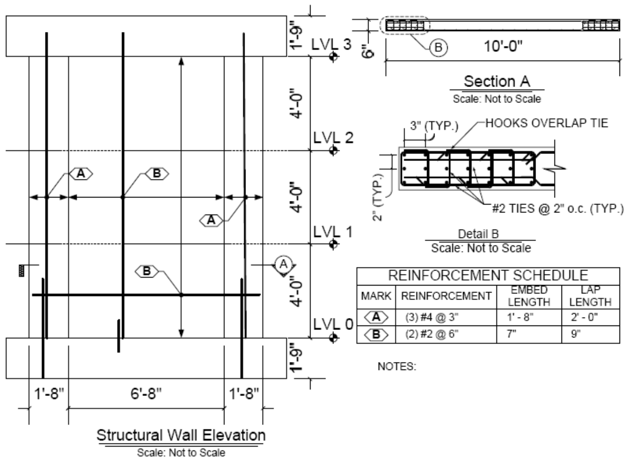


Fig. 5a NEESR UW wall tests (Lowes et al, 2011)

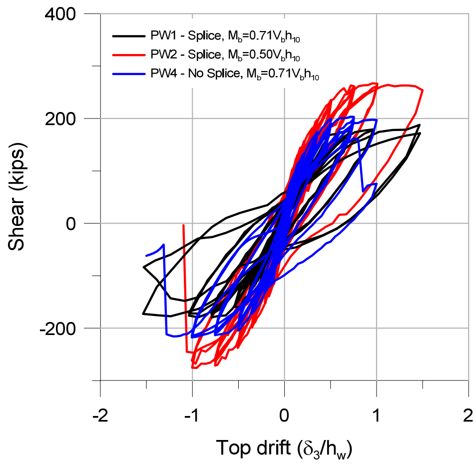


Fig. 5b Base Shear vs Drift

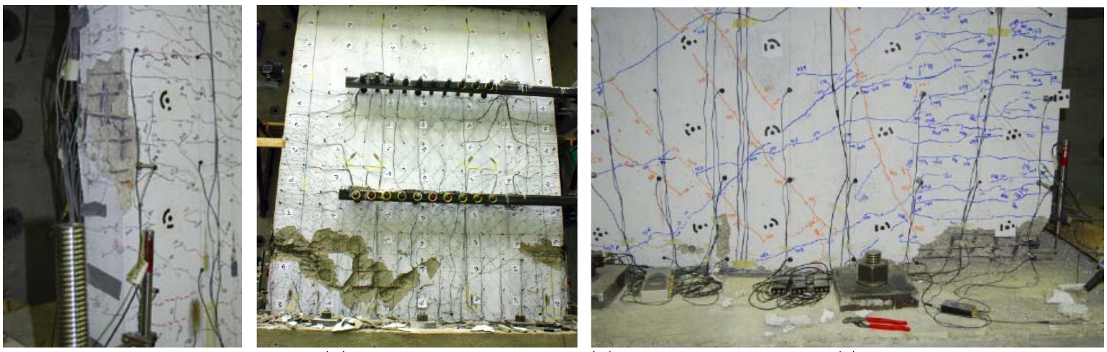


Figure 6 Wall damage: (a) PW2 @ 1.0% drift; (b) PW2 end of test; (c) PW4 @ 1.0% drift

Nagae, et al. (2011) reports E-Defense tests on two 4-story buildings, one conventionally reinforced and the other using high-performance RC construction, both with rectangular wall cross sections (Fig. 7a). The conventionally reinforced wall had confinement exceeding U.S. requirements, with axial load of approximately $0.03A_g f_c'$, yet the compression boundary zone sustained localized crushing and lateral buckling (Fig. 7b, following Kobe 100% motion). The base overturning moment versus roof displacement responses are plotted in Fig. 8; base rotations are slightly less than the roof drift ratio (e.g., for Kobe 100%, the base rotation measured over $0.27l_w$ is a little more than 0.02). Following crushing of boundary regions, sliding shear responses increased substantially during the Kobe 100% test (Fig. 8). Sliding displacements in the Takatori 60% test reached the limits of the sensor, +45mm and -60mm with peak shear of +/- 2000 kN. It is noted that the relatively large clear cover over the boundary longitudinal bars was used (~40mm) and the boundary transverse reinforcement was insufficient to maintain the boundary compressive load following cover spalling. It is noted that the crushing/spalling of the boundary region was accompanied by lateral buckling of the compression zone, as was observed in Chile and New Zealand (Fig. 2).

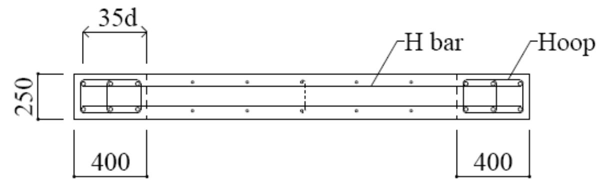
1Fl.	Section					
	B x D	2,500 x 250				
	Rebar	2 x 6-D19		Vertical	D13@300 (W)	
	Hoop	A	2,3-D10@80	Horizontal	A	D10@125 (W)
		C	2,3-D10@100		C	D10@200 (W)
	Joint	2,2-D10@150				

Figure 7a RC conventional wall (Nagae et al., 2011)

Figure 7b Wall damage

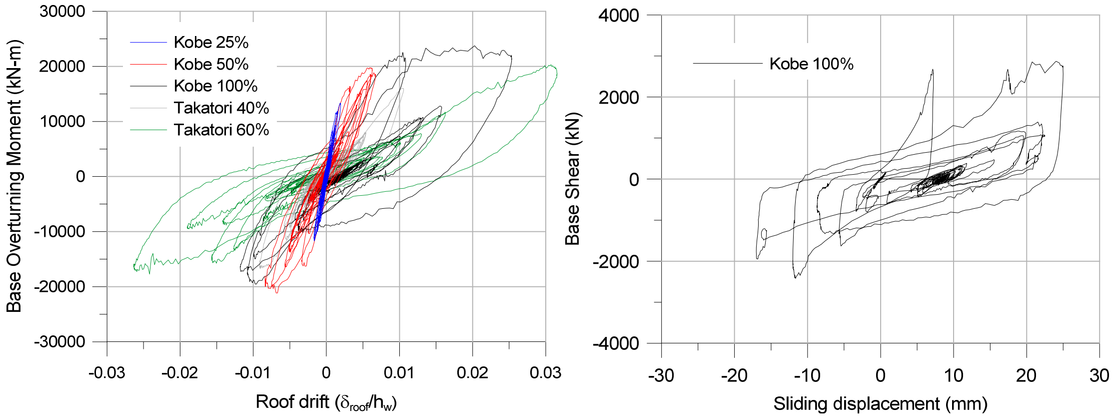


Figure 8 RC conventional building responses (structural wall direction)

Exploratory tests on prisms (Moehle, 2010) also showed a tendency for thin wall boundaries to buckle over an extended height of the wall (Fig. 9). Two buckling mechanisms may occur observed. If a wall segment is subjected to plastic tensile straining, the pre-cracked boundary zone becomes a relatively flexible element that might buckle globally under certain conditions. (This type of behavior was observed in past laboratory tests, and has been studied analytically – see Corley et al., 1981; Paulay and Priestley, 1993; Chai and Elayer, 1999). A second global buckling mode begins with spalling of cover concrete, leaving a relatively thin core with longitudinal reinforcement that tends to buckle laterally, displacing the remainder of the wall. As noted in the section on recent earthquake reconnaissance, the latter mode was widely observed in the 2010 Chile earthquake, and also for the E-Defense test. This latter buckling mode has not been studied previously.

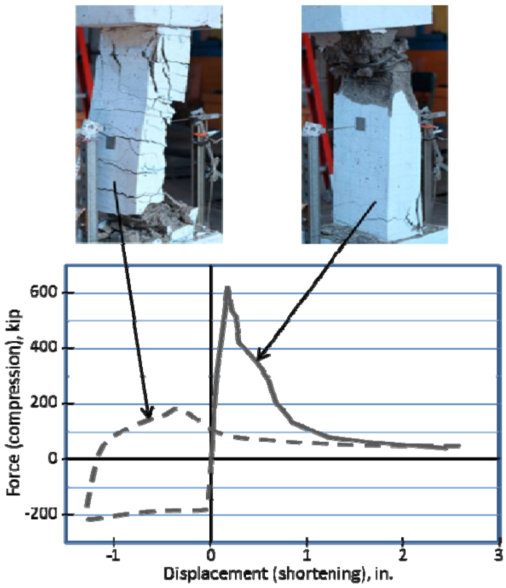


Figure 9 Exploratory prism tests

The pre-NEESR tests conducted at NEES@Minnesota (Brueggan, 2009; Waugh and Sritharan, 2010; Brueggen and French, 2010) studied the role of biaxial loading by subjecting cantilever walls with T-shaped cross sections to biaxial loading and comparing their results with similar tests subjected to in-plane loading (Thomsen and Wallace, 2004). The 6 in (152.4mm) thick walls exhibited rotations over the first story ($h_s = 0.8l_w$) of approximately 0.02 prior to lateral strength degradation. Their findings suggest that analytical models validated previously for in-plane loading adequately captured the measured responses for combined in- and out-of-plane loading. However, based on video and post-test observations, damage at wall boundaries of the conventional reinforced concrete building tested on the E-Defense shaking table may have been influenced by simultaneous in-plane and out-of-plane responses. The New Zealand Royal Commission report (NZRC, 2011) raises the issue of biaxial loading as a possible contributing factor to the unexpected wall damage in the February 2011 earthquake. This issue has not been adequately studied, and the issue is complicated by the observation that out-of-plane failures are observed at wall boundaries (and prisms) for in-plane loads alone.

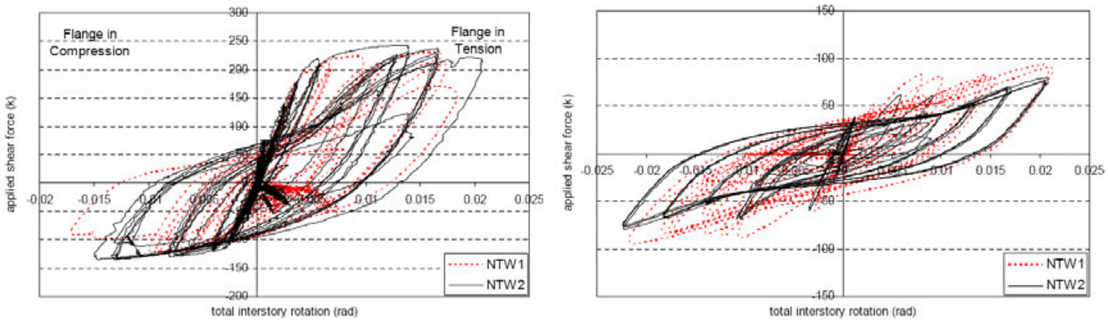
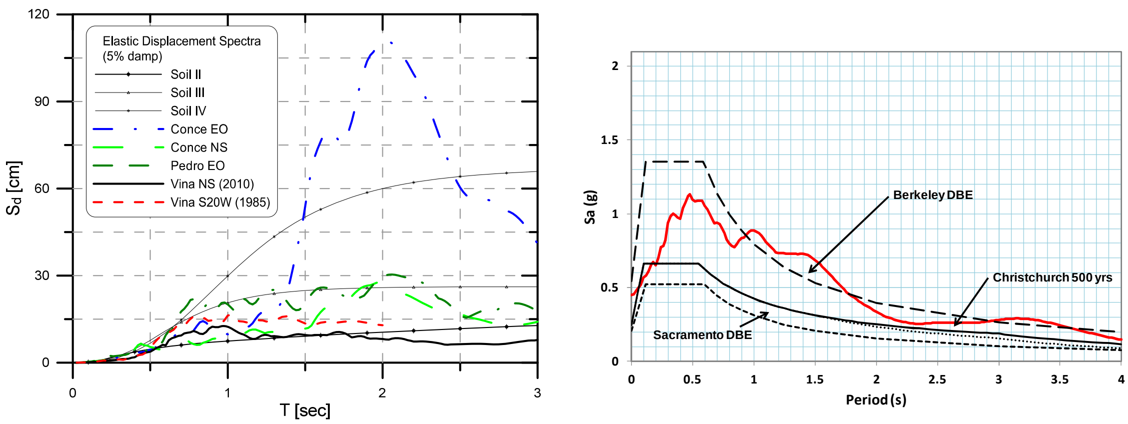


Fig. 10 Load vs displacement relations: (a) web direction; (b) Flange direction (Brueggen, 2009)

Recorded Ground Motions

Response Spectra computed using ground motions recorded in recent earthquakes have significantly exceeded values used for design. For example, spectra for records in Chile (Boroschek et al, 2010) and Christchurch (Elwood, 2011) significantly exceed values used for design (Fig. 11). For Chile, a vast majority of buildings are designed for the Soil II spectrum, whereas spectral ordinates are generally 2 to 6 times the values for Soil II over a broad period range. Given such large demands it is important to re-evaluate how displacement demands influence design requirements for structural walls.



(a) Chile displacement spectra (b) Christchurch acceleration spectra (Elwood, 2011)

Fig. 11 Spectra from recent large earthquakes

Summary

Wall performance in recent earthquakes and laboratory tests raise a number of design concerns. In Chile, brittle failures at wall boundaries were likely influenced by the level of axial stress (possibly leading to compression failures), the larger than expected displacement demands, the use of unsymmetric wall cross sections, and the lack of closely-spaced transverse reinforcement at wall boundaries. A particularly noteworthy aspect of recent tests (Nagae et al., 2011; Lehman and Lowes, 2011; Moehle et al., 2010) is the failure of relatively thin wall boundaries to develop ductile behavior in compression, even though they complied with ACI 318 special boundary element requirements, as well as Japan Standard Building Law and AIJ (2010) requirements. Recent tests to investigate the role of splices within the plastic hinge region of structural walls suggest that splices will substantially reduce wall inelastic deformation capacity. Given these observations, current ACI 318-11 code provisions for Special Structural Walls are reviewed to identify possible concerns and to suggest changes that could be implemented to address these concerns.

ACI 318 CHAPTER 21 CODE PROVISIONS FOR SPECIAL STRUCTURAL WALLS

Provisions for "Special Structural Walls" are contained in ACI 318-11 §21.9 and include provisions for Reinforcement (21.9.2), Shear Strength (21.9.4), Design for Flexural and Axial Loads (21.9.5), and Boundary Elements of Special Structural Walls (21.9.6). In light of the prior discussion, key aspects of these provisions are reviewed and areas of concern are noted. In many cases, insufficient information is available to develop comprehensive requirements and comments provided here are advisory.

Reinforcement and splices

A single curtain of web reinforcement is allowed if the wall shear stress is less than $0.17\sqrt{f_c' MPa}\ MPa$. This provision is acceptable for squat walls with low shear stress (e.g., walls with aspect ratio less than 1.5); however, for slender walls where buckling of boundary vertical reinforcement and lateral instability are more likely due to significant tensile yielding of reinforcement under cyclic loading, two curtains should always be used. This recommendation applies to both Special Structural Walls (high ductility) and Ordinary Structural Walls (moderate ductility).

Recent laboratory tests have identified that wall deformation capacity may be compromised in cases where splices exist within the wall critical section (plastic hinge) because nonlinear deformations are concentrated outside of the splice region, either at the wall-foundation interface (large moment gradient; Johnson, 2010) or above the splice (nearly uniform wall moment; Birely et al., 2010). Given these results, it is questionable whether boundary vertical reinforcement should be lapped spliced within the plastic hinge region. Test results did indicate that use of ACI 318-11 Type II couplers performed adequately. The option of staggering splices is not addressed here.

Design displacement and plastic hinge length

The model used to develop ACI 318-11 §21.9.6.2 provisions is shown in Figure 12. Given this model, the design displacement $\delta_u(ACI) \equiv \delta_x = C_d \delta_e / I\,(ASCE\ 7)$ is related to local plastic hinge rotation and extreme fiber compressive strain as:

$$\theta_p = \frac{\delta_u}{h_w};\quad \theta_p = \left(\phi_u = \frac{\varepsilon_c}{c}\right)\left(l_p = \frac{l_w}{2}\right) \quad \therefore \quad \varepsilon_c = 2\left[\frac{\delta_u}{h_w}\right]\left[\frac{c}{l_w}\right] \tag{1}$$

If the compressive strain exceeds a limiting value, typically taken as 0.003, then special transverse reinforcement is required. In ACI 318-11 Equation (21-8), this approach is rearranged to define a limiting neutral axis depth versus a limiting concrete compressive strain as:

$$c_{\text{limit}}=\frac{0.003 l_w}{2(\delta_u/h_w)}=\frac{l_w}{667(\delta_u/h_w)}\approx\frac{l_w}{600(\delta_u/h_w)} \tag{2}$$

In this approach, it is obvious that the result is sensitive to the values used for the design displacement and the plastic hinge length. Revised formulations, using a detailed displacement-based design approach (Wallace and Orakcal, 2002) and a plastic hinge length that varies with wall thickness (Wallace, 2011), produces the following relation:

$$\frac{\delta_u}{h_w}=\varepsilon_{cu}\left(\alpha\frac{t_w}{l_w}\frac{l_w}{c}\right)\left(1-\frac{\alpha}{2}\frac{t_w}{h_w}\right)+\frac{\varepsilon_{sy}}{(1-c/l_w)}\left(\frac{11}{40}\frac{h_w}{l_w}-\alpha\frac{t_w}{l_w}+\alpha^2\frac{t_w}{h_w}\frac{t_w}{l_w}\right) \tag{3}$$

Where t_w is the wall thickness, c is the neutral axis depth, h_w is the wall height, l_w is the wall length, and ε_{sy} is the tensile reinforcement yield strain. The constant 11/40 results based on the assumed distribution of lateral force over the height of the wall (Wallace and Moehle, 1992). In (3), the relationship between the wall neutral axis depth, concrete compressive strain, and drift is computed for various ratios of l_w/t_w and h_w/l_w for the three assumed values of plastic hinge length. For this preliminary study, wall aspect ratio h_w/l_w is set to 3.0 and the ratio of l_w/t_w is set to 13.3 for U.S. construction. Concrete compressive strain is set to 0.003; results presented in Fig. 13 define when special transverse reinforcement would be required at wall boundaries for three plastic hinge lengths.

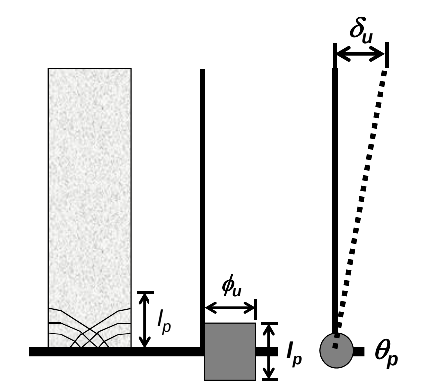


Fig. 12 ACI 318-11 §21.9.6.2 model

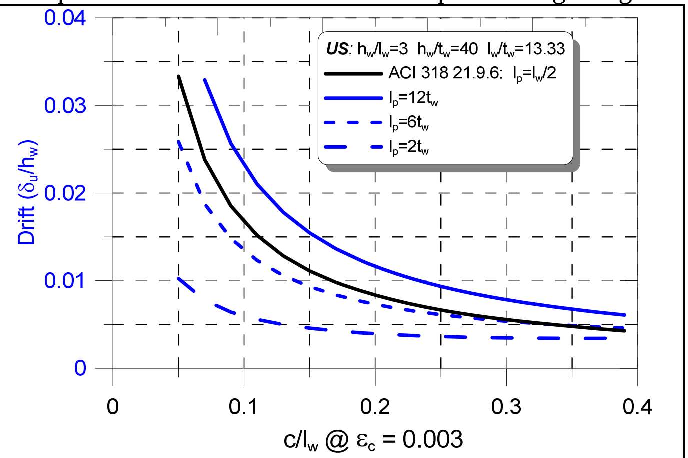


Fig. 13 Influence of plastic hinge length on need for SBEs

According to Fig. 13, if the drift ratio is 0.01, the neutral axis must exceed $0.17l_w$ before SBE are required by ACI 318-11. However, for the same neutral axis depth of $0.17l_w$, if inelastic deformations are concentrated over a short height $(l_p=(a=2)t_w)$, only less than one-half of this drift ratio (0.005), can be tolerated before SBEs are required. The sensitivity of the results suggests that measures are needed to ensure appropriate spread of plasticity by requiring walls to be tension-controlled or by ductile yielding of concrete in compression.

In current US codes the intent is to provide 90% confidence of non-collapse for MCE shaking. In contrast, the current ACI confinement trigger is based on 50% confidence of not exceeding the concrete crushing limit in the Design Basis Earthquake (which is much lower shaking intensity than the MCE). To address this issue, it is necessary to adjust the Equation (21-8) to be more consistent with the building code performance intent. Three factors need to be considered: 1) MCE exceeds DBE. 2) There is dispersion about the median response. 3) Damping is likely to be lower than the 5% value assumed in the ACI provisions. To address these issues, the coefficient of 600 in the denominator of Equation (21-8) in ACI 318-11 should be increased by a factor of approximately 1.5 to adjust to MCE level shaking and to consider dispersion, and by approximately 1.2 to 1.3 to account for potential lower damping ratios; therefore, a coefficient of 1000 to 1200 is more needed.

Axial load and compression-controlled walls:

As noted above, the provisions of 318-11 §21.9.6.2 assume that nonlinear deformations within the critical (plastic hinge) region of the wall will spread out over a distance equal to one half the member depth. ACI 318-11 §9.4 defines tension- and compression-controlled sections; however, no guidance is provided on how these requirements should be applied to special (or ordinary) structural walls. In addition, ACI 318 and ASCE 7 do not place limits on wall axial stress. The performance of walls in Chile suggests that higher axial stresses and wall cross section shape (e.g., T-shaped) may lead to cases where concrete compressive strain reaches 0.003 prior to yield of tension steel.

Various approaches could be used to address this issue, such as placing limit on axial stress or requiring wall critical sections to be tension-controlled. In the 1997 version of the Uniform Building Code, wall axial load was limited to $0.35P_0$; for higher axial loads the lateral strength and stiffness of the wall could not be considered. An alternative to neglecting the lateral-force-resistance of compression-controlled walls would be to impose more stringent design requirements, such as always requiring Special Boundary Elements (SBEs) for wall critical sections that are not tension-controlled, using the same definition of tension-controlled used for beams, i.e., where the reinforcement tensile strain exceeds 0.005. In addition, it also might be necessary to impose a larger minimum wall thickness (t_w) and a smaller wall slenderness ratio (h_s/t_w) for compression-controlled walls. The objective of these requirements would be to maintain a stable compressive zone as the concrete yields in compression.

Even with more stringent design requirements for compression-controlled wall sections, it may not be possible to expect significant inelastic deformation capacity (rotation) can be achieved through compression yielding. That is, it might be prudent to limit the nonlinear deformations. This objective can be accomplished by placing a limit on concrete compressive strain using Equation (1). For an assumed neutral axis depth of $0.6l_w$ (for balanced failure), a limiting compression strain of 0.01, Equation (1) gives: $\delta_u / h_w < 0.010/(2)(0.6) = 0.0083$. Given the simplifying assumptions associated with Equation (1), a slightly higher value (0.01) might be appropriate.

Boundary Element Detailing

ACI 318-11 detailing requirements for SBEs are based on requirements that were developed for columns; these provisions may be insufficient for SBEs of thin walls. The review of recent wall damage in earthquakes and laboratory tests provides sufficient evidence to raise concerns related to detailing of thin walls. For example, although the quantity of transverse reinforcement provided at the boundaries of the conventional RC wall tested at E-Defense were 1.4 and 2.1 times that required by ACI 318-11 §21.9.6.4 (for the larger spacing of 100mm used at Axis C), concrete crushing and lateral instability (Fig. 7b) occurred earlier in the Kobe 100% test, followed by substantial sliding (Fig. 8). Inspection of the damaged boundary zone revealed that relatively large clear cover was used, on the order of 40mm (larger than the code minimum in ACI 318, which is 19mm), suggesting that the confined core was incapable of maintaining stability of the compression zone following loss of concrete cover. For smaller columns, ACI 318-11 Equation (21-4), which is based on maintaining column axial load capacity after cover concrete spalling, typically governs the selection of transverse reinforcement for smaller columns where cover makes up a larger percentage of the gross concrete section. This equation also was required for wall SBEs prior to ACI 318-99; it was dropped because it rarely controlled for the thicker walls that were commonly used at that time. For the E-Defense conventional RC wall, the provided transverse reinforcement is only 0.34 and 0.45 times that required by ACI 318-11 Equation (21-4), suggesting that improved performance may have resulted had this relation been required. Additional testing is needed to determine if reinstating (21-4) is sufficient to ensure ductile behavior of thin boundary zones.

ACI 318-11 §21.6.6.2 allows the distance of 14” (356mm) between adjacent hoops or ties. Use of such a large spacing for thin SBEs is unlikely to provide sufficient confinement (Fig. 14) and is incompatible with use of a vertical spacing one-third the wall thickness. For example, for a 10 in.

(254mm) thick wall, such as used in the E-Defense test, the vertical spacing per ACI 318-11 is limited to 3.33" (84.6mm); however, the horizontal spacing along the wall can reach 356mm (356/84.6 = 4.2). An additional limit should be considered for wall SBEs, similar to that used for vertical spacing, where the horizontal distance between legs of hoops or ties is limited to a fraction of the wall thickness, e.g., two-thirds t_w. No allowing intermediate, unsupported bars at the wall edge, which initiated the section failure for test PW2 (Fig. 6a), also should be considered.

Wall Slenderness and Lateral Stability

To limit instability failures, limits on wall slenderness should be considered, similar to what was done in the Uniform Building Code (1997), which imposed a slenderness limit of $t_w \geq h_s/16$. Based on observations in recent earthquakes and tests, a lower limit should probably be used within plastic hinge zones, a ratio of $t_w \geq h_s/10$ was recently recommended in Moehle et al. (2011). This issue is currently under study by ATC 94 (2011).

CONCLUSIONS

Wall performance in recent earthquakes and laboratory tests is reviewed and American Concrete Institute 318 provisions are reassessed to identify possible shortcomings. The findings suggest a number of issues require more in-depth study, particularly for thin walls, as well as approaches that could be implemented to address these issues. In particular, changes are needed to increase the design displacement used in ACI 318-11 Equation (21-8), a factor of two is suggested, and to ensure spread of plasticity consistent with the derivation of Equation (21-8). To address this latter issue, walls should either be tension controlled or be designed and detailed to ensure ductile compression yielding by requiring that walls be thicker and by imposing a limit on wall slenderness. Limiting wall compression strain for compression-controlled walls also might be prudent.

ACKNOWLEDGMENTS

This research described in this paper was carried out with funding from various sources, including the EERI Learning from Earthquakes program (NSF CMMI-0758529), NSF RAPID projects to enhance US-Japan collaboration related to the E-Defense tests in December 2010 (CMMI-1110860 and CMMI-1000268; Program Director Joy Pauschke), NSF NEES REU (CMMI-0927178), as well as support provided to the first author by the Japan Society for the Promotion of Science (JSPS) Invitation Fellowship Program during the fall 2010. This support is gratefully acknowledged. The authors would like to thank those researchers who have contributed their research results to NEEShub, which provides an invaluable resource, as well as other researchers, including C French (U Minnesota), S. Sritharan (Iowa State), L Lowes and Dawn Lehman (U Washington), and K Elwood (U BC). And finally, the authors would like to express their deep appreciation to our Japanese collaborators within the reinforced concrete buildings area related to the December 2010 E-Defense tests for sharing their research ideas and results, including: T Nagae (NIED), K Tahara (NIED), T Matsumori (NIED), H Shiohara (U Tokyo), T Kabeyasawa (U Tokyo ERI), S Kono (Kyoto U), M Nishiyama (Kyoto U); M. Nakashima (NIED, Kyoto U). The opinions stated here are those of the authors and do not reflect Opinions, findings, conclusions, and recommendations in this paper are those of the authors, and do not necessarily represent those of the sponsors or other individuals mentioned here.

REFERENCES

American Concrete Institute. (1995). *Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95)*, Farmington Hills, MI.

American Concrete Institute. (1999). *Building Code Requirements for Structural Concrete (ACI 318-99) and Commentary (ACI 318R-99)*, Farmington Hills, MI.

American Concrete Institute. (2011). Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary *(ACI 318R-11)*, Farmington Hills, Michigan.
American Society of Civil Engineers. (2005). ASCE7-05: Minimum Design Loads for Buildings and Other Structures, Reston, VA.
Architectural Institute of Japan (2010), "Standard for Structural Calculation of Reinforced Concrete Structures," first published in 1933.
ATC-94 (2011). Analysis of Seismic Performance of Reinforced Concrete Buildings in the 2010 Chile Earthquake, Including Effects of Non-Seismic-Force-Resisting Building Structural Elements - Task Order 21, https://www.atcouncil.org/Projects/nehrp-jv.html.
Birely A, Lehman D, Lowes L, Kuchma D, Hart C, Marley K, "Investigation of the Seismic Behavior and Analysis of Reinforced Concrete Structural Walls, Proceedings, 14th World Conference on Earthquake Engineering, Beijing, China, October 12-17, 2008.
Birely A, Lehman D, Lowes L, Kuchma D, Hart C, Marley K, "Investigation of the Seismic Response of Planar Concrete Walls," *Proceedings" Paper 773*, 9th U.S. National Conference and 10th Canadian Conference on Earthquake Engineering, July 25-29, 2010, Ontario, Canada.
Boroschek R., Soto P, and Leon R, 2010. Registros del Terremoto del Maule Mw=8.8 27 de Febrero de 2010, *Red Nacional de Acelerógrafos del Departamento de Ingeniería Civil*, Facultad de Ciencias Físicas y Matemáticas, Universidad de Chile, Informe *RENADIC 10/05*, 100 pp. (http://www.terremotosuchile.cl/)
Brueggen BL (2009), "Performance of T-shaped Reinforced Concrete Structural Walls under Multi-Directional Loading," PhD Dissertation, University of Minnesota, Department of Civil Engineering, 498 pp.
Brueggen BL, French CW, "Simplified Modeling of Non-Rectangular RC Structural Walls," *Proceedings" Paper 1713*, 9th U.S. National Conference and 10th Canadian Conference on Earthquake Engineering, July 25-29, 2010, Ontario, Canada.
Chai YH and Elayer DT (1999). "Lateral Stability of Reinforced Concrete Columns under Axial Reversed Cyclic Tension and Compression," ACI Structural Journal, ACI, V. 96, No. 5, pp. 780-789.
Corley WG, Fiorato AE, and Oesterle RG (1981). "Structural Walls," Publication SP-72, ACI, pp. 77-131.
EERI (2010). "The Mw 8.8 Chile Earthquake of February 27, 2010," EERI Special Earthquake Report, Earthquake Engineering Research Institute, June.
EERI (2011). "NEES/E-Defense Collaboration on Tests," *EERI Newsletter*, Earthquake Engineering Research Institute, V. 45, No. 2.
EERI (2011). "The M 6.3 Christchurch, New Zealand, Earthquake of February 22, 2011," EERI Special Earthquake Report, May 2011.
Elwood KJ, (2011), Personal Communication. [see Also EERI Christchurch Earthquake Clearing House: http://www.eqclearinghouse.org/2011-02-22-christchurch/].
Johnson B (2010), "Anchorage Detailing Effects on Lateral Deformation Components of R/C Shear Walls, MS Thesis, University of Minnesota, Department of Civil Engineering, 353 pp.
Kabeyasawa, T., Kabeyasawa, T., Kim Y. (2010a), "Collapse Simulation of Reinforced Concrete Buildings with ASFI Approach, *Proceedings" Paper 816*, 9th U.S. National Conference and 10th Canadian Conference on Earthquake Engineering, July 25-29,2010, Ontario, Canada.
Kabeyasawa, T., Kabeyasawa, T., Kim Y., Kabeyasawa T, Bae K, Quang PV, "Strength and Deformability of Reinforced Concrete Columns with Wing Walls (2010b)," *Proceedings, Paper 813*, 9th U.S. National Conference and 10th Canadian Conference on Earthquake Engineering, July 25-29, 2010, Ontario, Canada.
Kabeyasawa, T., Kabeyasawa, T., Matcumori, T., Kabeyasawa, T. and Kim, Y. (2008). "Full-scale dynamic collapse tests of three-story reinforced concrete buildings on flexible foundation at E-defense," *Proceedings of the 14th World Conference on Earthquake Engineering*, Beijing, China, Oct. 2008, Paper ID: S15-002.
Lehman DE and Lowes LN (2011). Personal communication about laboratory test results for NEESR project Seismic Behavior, Analysis, and Design of Complex Wall Systems. [see also

nees.org/warehouse/project/104]
Massone LM, Wallace JW (2011), “Lessons from Chile: Impacts of Earthquake Engineering of RC Buildings in the US,” EERI/NEES Webinar. http://nees.org/resources/3192
Moehle JP, Acevedo C, and Creagh A (2010). “Exploratory tests of wall boundary elements subjected to alternating tensile and compressive loadings,” Poster and oral presentations at 2010 PEER Annual Meeting.
Moehle, J.P., T. Ghodsi, J.D. Hooper, D.C. Fields, and R. Gedhada (2011). “Seismic Design of Cast-in-Place Concrete Special Structural Walls and Coupling Beams: A guide for practicing engineers,” *NEHRP Seismic Design Technical Brief No. 6*, National Institute of Standards and Technology, Gaithersburg, MD.
Nagae T, Tahara K, Matsumori T, Shiohara H, Kabeyasawa T, Kono S, Nishiyama M (Japan) and Wallace JW, Ghannoum W, Moehle JP, Sause R, Keller W , Tuna Z., “Design and Instrumentation of the 2010 E-Defense Four-Story Reinforced Concrete and Post-Tensioned Concrete Buildings,” Report No 2011-XX, Pacific Earthquake Engineering Research Center, Department of Civil & Environmental Engineering, University of California, Berkeley, Febraury 2011, 235 pp.
Naish D, Wallace JW, "Testing and Modeling of Diagonally-Reinforced Reinforced Concrete Coupling Beams," Proceedings, Special Session, 9th U.S. National Conference on Earthquake Engineering, Toronto, Canada, July 2010 (Paper 1575).
NCh 433.Of96 (1996). Diseño Sismico de Edificios (Chilean Building Code), Chile.
NZRC (2011). “Canterbury Earthquakes Royal Commission – Interim Report,” http://canterbury.royalcommission.govt.nz/Interim-Report.
Panagiotou, M., and Restrepo, J.I., (2007). “Practical Lessons Learned from the Full-Scale 7-Story Building Shake Table Test at UC San Diego,” 2007 SEAOC Convention, Squaw Creek, CA, Sept. 26-29, 2007, pp. 57-74.
Parra-Montesinos G, Wight JK, Lequesne RD, Seekit M, “A summary of ten years of research on HPFRC coupling beams,” High Performance Fiber Reinforced Cement Composites 6, Parra-Montesinos, Gustavo J., Reinhardt, Hans W., Naaman, Antoine E. (Ed.), 2012, 560 p.
Paulay, T., and M.J.N. Priestley (1993). “Stability of Ductile Structural Walls,” *ACI Structural Journal*, American Concrete Institute, V. 90, No. 4, pp. 385-392.
Sriram A, Sritharan S, “Nonlinear Fiber-Based Analysis of Rectangualr Concrete Walls Designed with Different Anchorage Details,”, *Proceedings, Paper 123*, 9th U.S. National Conference and 10th Canadian Conference on Earthquake Engineering, July 25-29,2010, Ontario, Canada.
Thomsen IV, JH and Wallace JW (2004), “Displacement-Based Design of Slender RC Structural Walls – Experimental Verification, J. Struct. Eng., ASCE, Vol. 130, No. 4, pp 618-630.
UBC (1997). *Uniform Building Code*, International Council of Building Code Officials, Whittier, CA.
Wallace JW (1996), “Evaluation of UBC-94 Provisions for Seismic Design of RC Structural Walls,” Earthquake Spectra, Vol. 12, No. 2, pp 327 – 348.
Wallace JW (2011), “February 27, 2010 Chile Earthquake: Preliminary Observations on Structural Performance and Implications for U.S. Building Codes and Standards,” ASCE Structures Congress, Paper 1159, Las Vegas, April 14-16, 2011.
Wallace JW, Moehle JP (1992), “Ductility and Detailing Requirements of Bearing Wall Buildings,” J. Struct. Eng., ASCE, Vol. 118, No. 6, pp.1625-1644.
Wallace JW, Orakcal K. (2002), “ACI 318-99 Provisions for Seismic Design of Structural Walls,” ACI Structural Journal, Vol. 99, No. 4, American Concrete Institute, pp. 499-508.
Waugh J, Aaleti S, Sritharan S, Zhao J, "Nonlinear Analysis of Rectangular and T-Shaped Concrete Walls," ISU-ERI-Ames Report ERI-09327, Iowa State University, Department of Civil, Construction and Environmental Engineering, December 2008, 351 pp.
Waugh JD, Sritharan S (2010), “Nonlinear Analysis of T-Shaped Concrete Walls Subjected to Multi-Directional Loading, *Proceedings, Paper 1506*, 9th U.S. National Conference and 10th Canadian Conference on Earthquake Engineering, July 25-29,2010, Ontario, Canada.