Performance of Structural Walls in Recent Earthquakes and Tests and Implications for US Building Codes

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SUMMARY
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. 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 laboratory tests also have demonstrated inadequate performance in some cases, indicating a need to review code provisions, identify shortcomings and make necessary revisions.

Keywords: structural wall, earthquake damage, testing, codes, ACI 318, detailing

1. 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 as they produced 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 lateral 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 noteworthy aspect of these recent tests is the failure of relatively thin wall boundaries to develop ductile behavior, even though they complied with building code provisions and recommendations of ACI and AIJ.

Wall damage observed in recent earthquakes and laboratory tests strongly suggest that the problems noted 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.

2. OBSERVED PERFORMANCE

2.1 Wall Damage Observed in Recent Earthquakes

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, as buckling of vertical bars led to concentration of damage. Closer inspection of the wall boundary regions (Fig. 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 likely 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., Wall TW2 tested by 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. Laboratory testing is required to understand these behaviors; studies are underway as part of the ATC 94 project.

The 2011 Christchurch earthquake (EERI, 2011; NZRC, 2011) shows 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 Fig. 3 have either T-shaped (Fig. 2, 3b) or L-shaped (Fig. 3a) cross sections, which lead to large cyclic tension and compressive demands at the wall web boundary (Wallace, 1996). The wall web boundaries are 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, wall slenderness, or the ratio of the core thickness to the cover thickness.

2.2 Wall Damage observed in Recent laboratory Tests

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 and Sritharan, 2010; Brueggen and French, 2010), walls with couplers and splices in the plastic hinge region (Johnson, 2010, Birely et al., 2010), walls with higher shear demands (Birely et al., 2010; Tran and Wallace, 2012).

Johnson (2010) reports test results of isolated, slender (h_w/l_w and $M_u/V_ul_w \approx 2.67$) cantilever walls to investigate the behavior of anchorage details for flexural (vertical) reinforcement. Three walls were tested, 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 20ft (6.1m) above the bases. 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 ρ_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 ρ_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.

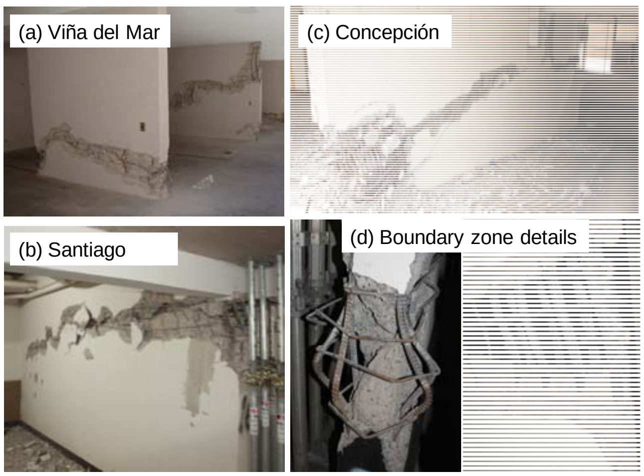


Figure 1 Typical wall damage in Chile Earthquake

Figure 2 Wall lateral instability

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

Figure3b Specimen TW2 (Thomsen and Wallace, 2004)

Lateral load versus top lateral displacement relations for RWC and RWS are plotted in Fig. 4a; results for RWC and RWN are very similar. For RWC, the wall reached rotations exceeding +0.035 (#5 in tension) and -0.02 (#9 in tension), whereas for RWS, the wall reached rotations of approximately +0.02 (#5 in tension) and -0.012 (#9 in tension). Damage was concentrated at the foundation-wall interface, with rotations at the interface accounting for about 0.015 of the top rotation of 0.02. Significant horizontal cracking between the boundary zones, was observed for specimens RWN and RWC suggesting that the quantity (and large spacing) vertical web reinforcement was insufficient to restrain opening of large horizontal cracks along the web. Damage concentrated at the foundation-wall interface for specimen RWS (Fig. 4b). The test results do indicate adequate performance in the case of the coupler and that the presence of the splice significantly reduced the wall lateral deformation capacity.

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=0.71$h_w V_b$), W2 (splice, M_b=0.50$h_w V_b$), and W4 (no splice, M_b=0.50$h_w V_b$). 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.

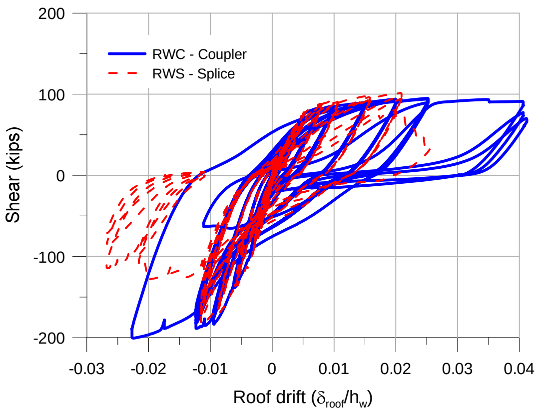


Figure 4a Load-displacement relations

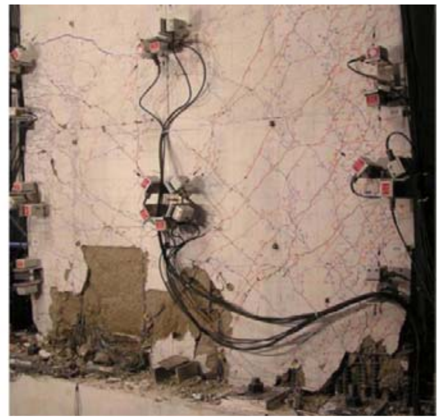


Figure 4b Wall damage at end of test (RWS)

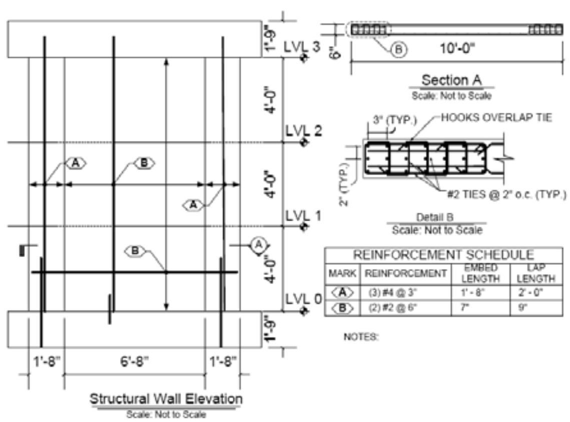


Figure 5a NEESR UW wall tests (Lowes et al, 2011)

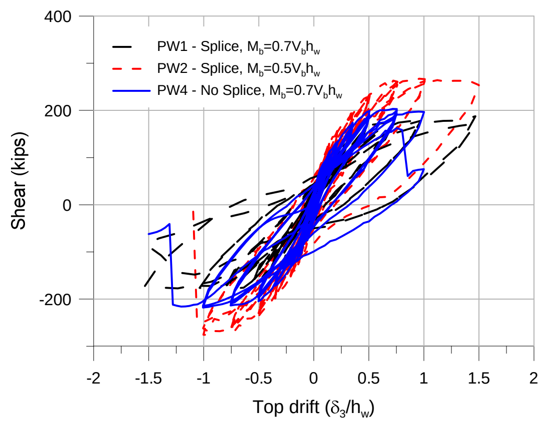


Figure 5b Base Shear vs Drift

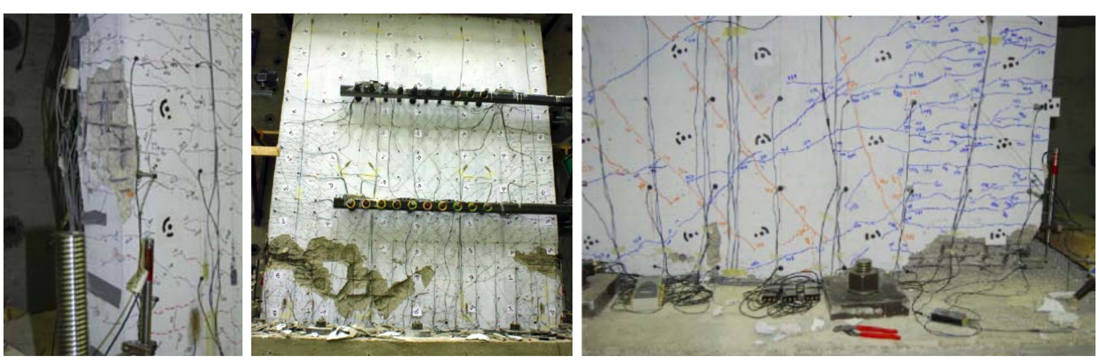


Figure 6 Wall damage: (a) PW2 @ 1.0% drift; (b) PW2 end of test; (c) PW4 @ 1.0% drift

Nagae, et al. (2011) summarizes important specimen details for NIED (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 US 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 sensor stroke limits, +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 using Equation (21-4) of ACI 318-11 (not required for walls). 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). It not clear what role biaxial loading had on the observed wall damage, this issue is still being studied; however, it is plausible that the susceptibility of the wall to lateral instability was impacted by biaxial loading.

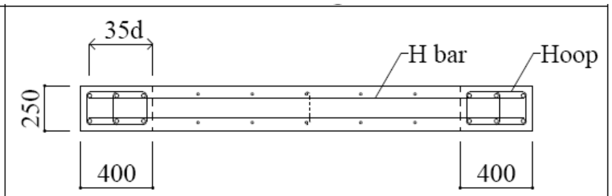
1Fl.	Section				
	B x D	2,500 x 250			
	Rebar	2 x 6-D19		Vertical	D13@300 (W)
	Hoop	A	2,3-D10@80	Horizontal	A D10@125 (W)
		C	2,3-D10@100		C D10@200 (W)
	Joint	2,2-D10@150			

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

Figure 7b Wall damage

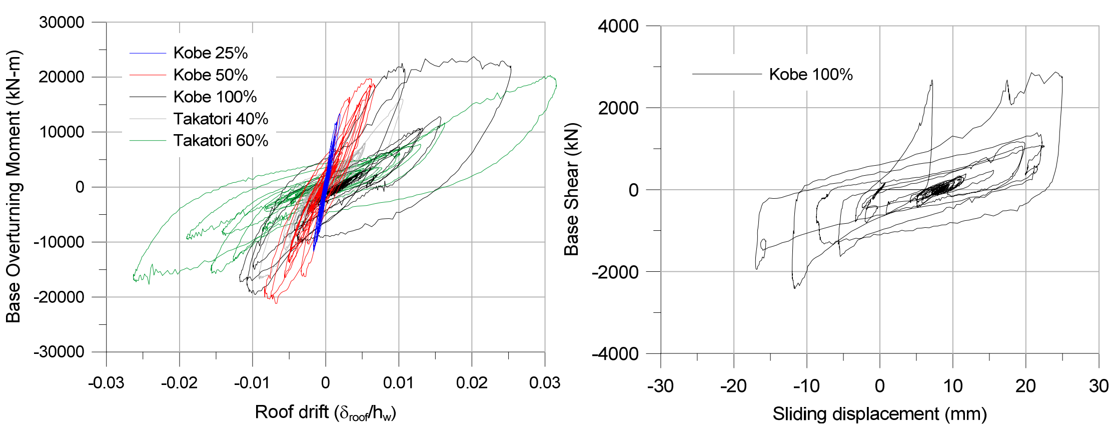


Figure 8 RC conventional building responses (structural wall direction)

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. 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.

2.3 Recorded Ground Motions

Spectral ordinates computed using ground motions recorded in recent earthquakes have significantly exceeded values used for design (Fig. 10). For Chile, a vast majority of buildings were designed for the SII spectrum, whereas spectral ordinates were generally 2 to 3 times values for SII over a broad period range. Similar observations apply to Christchurch. Given such large demands, it is important to re-evaluate how displacement demands influence design requirements for structural walls as well as the consequence of exceeding values used to assess detailing requirements at wall boundaries.

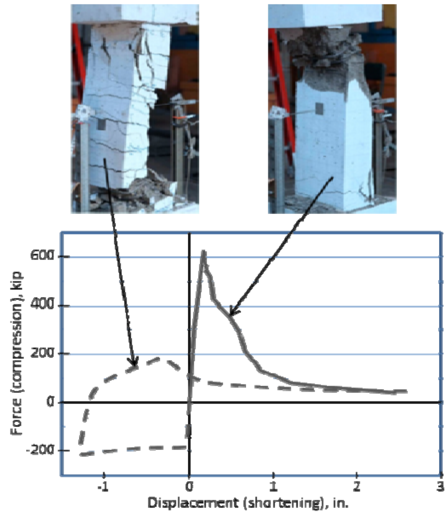


Figure 9 Exploratory prism tests

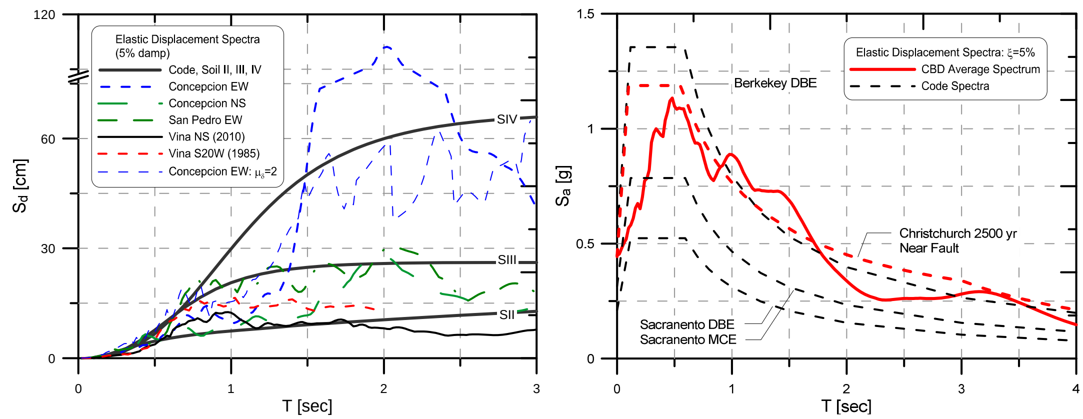


Figure 10 Spectra from recent large earthquakes: (a) Chile; (b) Christchurch

3. ACI 318 CHAPTER 21 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 preceding discussion, key aspects of these provisions are reviewed and areas of concern are noted.

3.1 Reinforcement and splices

A single curtain of web reinforcement is allowed in ACI 318-11 if the wall shear stress is less than $0.17\sqrt{f_c'}\ MPa$. This provision is probably acceptable for squat walls with low shear stress; however, for slender walls ($h_w/l_w > 2.0$); where buckling of boundary vertical reinforcement and wall lateral instability are more likely due to significant tensile yielding of reinforcement under cyclic loading, two curtains should always be used.

Recent laboratory tests have identified that wall deformation capacity may be compromised in cases where splices exist within the wall critical section (plastic hinge region) because nonlinear deformations are concentrated outside of the splice region. Given these results, it is questionable whether boundary vertical reinforcement should be lapped spliced within the plastic hinge region. If vertical boundary bars are spliced, the special detailing should be continued above the splice a distance equal to an upper bound estimate of the plastic hinge length. Test results did indicate that use of ACI 318-11 Type II couplers performed adequately. The option of staggering splices is not addressed here.

3.2 Design displacement and plastic hinge length

The model used to develop ACI 318-11 §21.9.6.2 provisions is shown in Figure 11. 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 θ_p and extreme fiber compressive strain ε_c as:

$$\theta_p = \frac{\delta_u}{h_w}; \quad \theta_p = \left(\phi_u = \frac{\varepsilon_c}{c}\right)\left(l_p = \frac{l_w}{2}\right) \quad \therefore \quad \varepsilon_c = 2\left[\frac{\delta_u}{h_w}\right]\left[\frac{c}{l_w}\right] \tag{3.1}$$

Where l_p is the plastic hinge length, h_w is the wall height, c is the neutral axis depth for (M_n, $P_{u,max}$) , and l_w is the wall length. 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 instead of a limiting concrete compressive strain as:

$$c_{\text{limit}} = \frac{0.003 l_w}{2\left(\delta_u / h_w\right)} = \frac{l_w}{667\left(\delta_u / h_w\right)} \approx \frac{l_w}{600\left(\delta_u / h_w\right)} \tag{3.2}$$

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, 2011; 2012), produces the following more comprehensive relation:

$$\frac{\delta_u}{h_w} = \varepsilon_{cu}\left(\alpha \frac{t_w}{l_w}\frac{l_w}{c}\right)\left(1 - \frac{\alpha}{2}\frac{t_w}{h_w}\right) + \frac{\varepsilon_{sy}}{\left(1 - c/l_w\right)}\left(\frac{11}{40}\frac{h_w}{l_w} - \alpha\frac{t_w}{l_w} + \alpha^2 \frac{t_w}{h_w}\frac{t_w}{l_w}\right) \tag{3.3}$$

where t_w is the wall thickness, 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 (3.3), if wall aspect ratio h_w/l_w is set to 3.0 and ratio of l_w/t_w is set to 13.3, which are fairly typical for U.S. construction, and concrete compressive strain is set to 0.003, the need for SBEs can be evaluated as a function of the assumed plastic hinge length (Fig. 12). For the ratio of l_w/t_w selected (13.33), α=6 is equivalent to l_p = 0.45l_w, or about the same value of 0.5l_w assumed in the development of ACI 318-11 relations in Equation (3.2). Special transverse reinforcement is required at wall boundaries for values above and to the right of the relations plotted in Figure 12. The sensitivity of the results presented in Figure 12 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 for compression-controlled walls. These issues are not currently addressed in ACI 318-11.

In current US codes, the intent is to provide 90% confidence of non-collapse for MCE shaking. In contrast, the current ACI confinement trigger (Equation 3.2) 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 ACI Equation (21-8), also Equation (3.2) in this paper, 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 ACI Equation (21-8) in should be increased to 1200 (Wallace, 2012).

3.3 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 length. 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. Wall TW1 tested by Thomsen and Wallace (2004) generally displays this behavior for flange in tension.

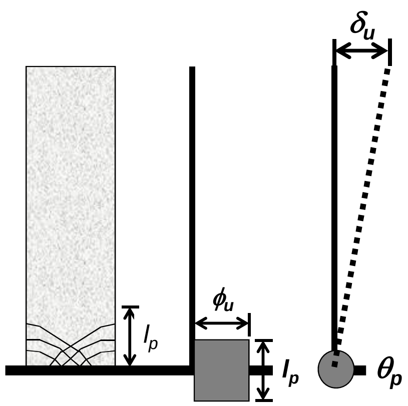


Figure 11 ACI 318-11 §21.9.6.2 model

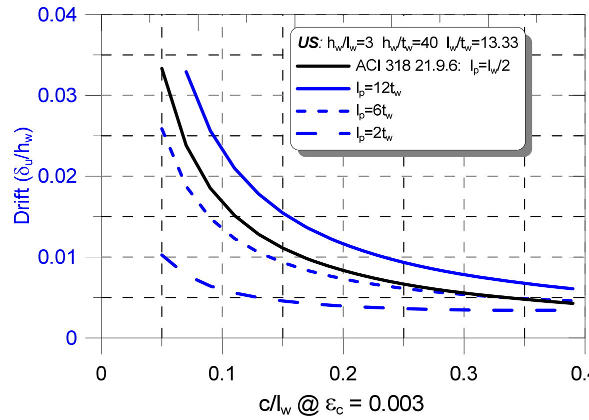


Figure 12 Influence of plastic hinge length on SBEs

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 SBEs to maintain a stable compressive zone as the concrete yields in compression. Even with more stringent design requirements, it might be prudent to place a limit on concrete compressive strain, e.g., 0.01, as it is not prudent to expect significant inelastic deformation capacity (rotation) can be achieved through compression yielding. This objective can be accomplished using displacement-based design using Equation (3.1). For $c/l_w \geq 3/8$, the value at which a section is roughly no longer tension-controlled per ACI 318-08 9.4, Eq. (3.1) gives: $(\delta_u / h_w)_{\text{limit}} = 0.010 l_w / (2 * 3 l_w / 8) = 0.0133$, whereas for $c/l_w \geq 0.6$, where a section is compression-controlled per ACI 318-08 9.4, Equation (3.1) gives: $(\delta_u / h_w)_{\text{limit}} = 0.010 l_w / (2 * 0.6 l_w) = 0.0083$. If the drift limit is exceeded, then redesign of the wall section would be required.

3.4 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 was 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 a 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. 13) 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., $2/3t_w$ or a value less than 356mm, e.g., 200mm. Not allowing intermediate, unsupported bars at the wall edge (Fig. 13), which initiated the section failure for test PW2 (Fig. 6a), also should be considered.

3.5 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 $t_w \geq h_s/16$. Lateral instability failures at wall boundary regions are influenced by a number of factors, including: slenderness, cross section shape, quantity of vertical reinforcement, detailing, axial load, design displacement, and load history. Introduction of a limit based on slenderness alone is unlikely to provide a robust solution to this problem; however, until a comprehensive study is available, use of $l_u/b \leq 16$ is recommended, although this limit may not be sufficient to preclude lateral instability failures for asymmetric wall cross sections (T- or L-shaped sections), where a lower limit of $l_u/b \leq 10$ might be appropriate at the web boundary opposite the flange given the large cyclic demands that may occur at this location (Wallace, 2012). This issue is currently under study by ATC 94 (2011).

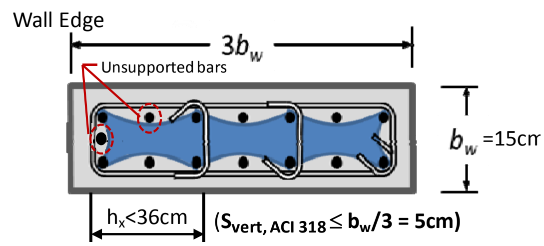


Figure 13 Confinement of thin wall sections

4. 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 that 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 multiplier of two on the design displacement is suggested, To ensure spread of plasticity consistent with the derivation of Equation (21-8), walls should either be tension controlled or be designed and detailed to ensure ductile compression yielding in compression. Limiting wall compression strain for compression-controlled walls also might be prudent, a limit of $(\delta_u / h_w)(c/l_w) \leq 0.005$ is suggested. Finally, reintroducing a limit on slenderness, e.g., $t_w \geq h_s/16$ is recommended, along with commentary to note that a lower ratio many be needed to avoid lateral instability at the web boundaries of flange walls.

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