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ACI 318-99 Provisions for Seismic Design of Structural Walls

by John W. Wallace and Kutay Orakcal

New provisions for evaluating flexural strength and detailing requirements at boundaries of reinforced concrete structural walls were incorporated into the 1999 version of the American Concrete Institute (ACI) Building Code, ACI 318-99. The ACI 318-99 provisions apply to both slender and stout walls, and walls with openings. The moment capacity of wall cross sections is based on a strain compatibility analysis, and two approaches are provided to determine whether specially detailed boundary elements are required. The first approach can be applied to all wall sections and involves checking a 0.2 f_c' stress limit at the wall boundary for code-specified loads. Although the stress limit is the same as that in ACI 318-95, significant changes were incorporated to address commonly identified shortcomings associated with the ACI 318-95 provisions. For the second approach, once the critical section along the wall height is identified, a simplified displacement-based design procedure is used to assess whether special detailing is needed. Background and illustrative examples are presented to demonstrate important concepts and appropriate use.

Keywords: design; reinforced concrete; shear; wall.

INTRODUCTION

Significant efforts have been expended in recent years to develop technically sound and widely applicable design provisions for reinforced concrete structural walls. These efforts were initiated to take advantage of the development of displacement-based design techniques for structural walls (Moehle and Wallace 1989; Wallace and Moehle 1992; Wallace 1994), as well as to address the shortcomings of the ACI 318-95 (ACI Committee 318 1995) provisions.

Wallace and Moehle (1992) identified that, for a majority of commonly used building and structural wall configurations, the stress-based approach used in ACI 318-89 to assess detailing requirements at wall boundaries was overly conservative for the design of slender walls. A displacement-based design methodology was developed (Moehle and Wallace 1989, 1994; Wallace and Moehle 1992) and experimental studies were conducted to validate the approach (Thomsen and Wallace 1995; Wallace 1996; Taylor, Thomsen, and Wallace 1996; Taylor, Cote, and Wallace 1998). Based on this research, as well as prior research (for example, Paulay 1986), the Structural Engineers Association of California (SEAOC) developed new shearwall design provisions that were first incorporated into the 1994 UBC (Uniform Building Code 1994) and then updated in the 1997 UBC (Uniform Building Code 1997). The 1994 and 1997 UBC provisions rely on the use of displacement-based design to assess the need for detailing at wall boundaries, and are valid for slender cantilever walls with a single critical section at the base of the wall. Supporting documentation provided in the SEAOC Blue Book (Structural Engineers Association of America 1996) is helpful in guiding the engineer through the relatively complex process. A discussion of the provisions, and some of the limitations, is provided by Wallace (1996). Although changes to the wall provisions were made to the UBC, provisions in the ACI 318 building code remained essentially unchanged in the 1995 version.

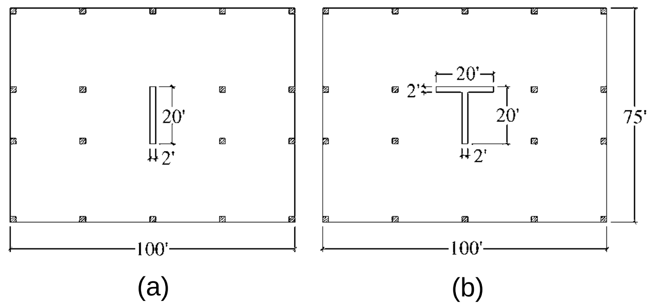

Fig. 1—Plan view of the building: (a) with rectangular wall; and (b) with T-shaped wall.

New provisions for evaluating detailing requirements at the boundaries of structural walls were adopted for ACI 318-99. The provisions were developed within ACI Subcommittee 318-H to incorporate displacement-based design and capacity-design concepts, as well as to address the shortcomings of ACI 318-95 and UBC-97 provisions. The objective of this paper is to provide detailed background information with simple illustrative examples to aid engineers in properly applying the new provisions. The background information and illustrative examples are presented parallel to each other to clarify important issues.

DESIGN OF REINFORCED CONCRETE STRUCTURAL WALLS

In the following sections, the design provisions for structural walls in ACI 318-95 and ACI 318-99 are reviewed. A five-story building with 12 ft (3.65 m) story heights is used to illustrate important concepts. The footprint of the building is 100 x 75 ft (30.48 x 22.86 m), and the floor dead and live loads are 150 and 40 lb/ft^2 (7.18 and 2.39 kPa), respectively. For the purposes of this discussion, only loads in the north-south direction are considered; therefore, a single wall is used to provide lateral force resistance in this direction. Although a single building geometry is used for this presentation, two different wall cross sections are evaluated to address a broad range of wall behavior and design issues—a rectangular wall cross section (denoted as Wall R) and a T-shaped wall cross section (denoted as Wall T). The wall web dimensions were initially selected to be 20 ft long by 2 ft thick (6.10 by 0.61 m) for Walls R and T, with a 20 ft-long by 2 ft-thick flange (6.10 by 0.61 m) for Wall T (Fig. 1). Determination of design

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John W. Wallace, *FACI, is an associate professor of civil engineering at UCLA, Los Angeles, Calif. He is a member of ACI Committee 318, Structural Concrete Building Code; 335, Composite and Hybrid Structures; 369, Seismic Repair and Rehabilitation; 374, Performance-Based Seismic Design of Concrete Buildings; and Joint ACI-ASCE Committee 352, Joints and Connections in Monolithic Concrete Structures. His research interests include response and design of buildings and bridges to earthquake actions, and laboratory and field testing of structural components and systems.*

Kutay Orakcal *is a PhD student in the Department of Civil Engineering, UCLA. He received his BS from Middle East Technical University in 1998, and his MS in civil engineering from UCLA in 2001. His research interests include behavior and modeling of reinforced concrete and steel-reinforced concrete elements and systems.*

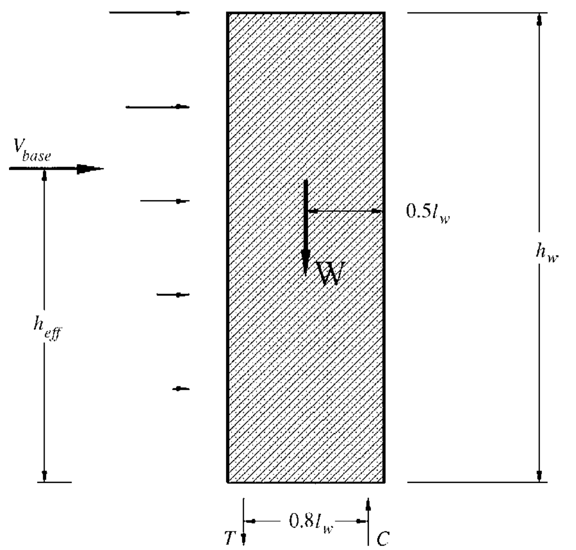

Fig. 2—Simplified model to estimate boundary reinforcement.

lateral forces and general modeling issues are presented for each wall, followed by detailed design and evaluation.

Preliminary design

Lateral earthquake forces were determined using UBC-97 requirements (Uniform Building Code 1997). For a five-story building located in Zone 4 (I = 1, Z = 0.4, S_B, Source Type A, 10 km, $T = C_t(h_n)^{3/4}$= 0.43 s, R = 5.5), UBC-97 Equation (30-5) controls the selection of base shear of 0.18W. The seismic dead load W is 5625 kips (25 MN) for a dead load of 150 lb/ft^2 (7.18 kPa); therefore, the base shear is 1000 kips (4.45 MN). The distribution of the lateral force over the height of the building increases linearly with values of 67, 133, 200, 267, and 333 kips (300, 592, 890, 1190, and 1480 kN) at the first through fifth levels, respectively.

To simplify the analysis, the wall was assumed to resist all of the lateral story forces; therefore, the wall base moment was M = 44,000 ft-kips (59.6 MN-m) and the effective height h_{eff} of the resulting lateral load was 44,000 ft-kips/1000 kips = 44 ft (13.4 m), or 73% of the total building height of 60 ft (18.3 m). A tributary area of 1125 ft^2 (104.5 m^2) was assumed for the wall at each level; therefore, the tributary dead and live loads at the base of the wall were 845 and 90 kips (3660 and 400 kN), respectively (with a live-load reduction of 60%).

Wall reinforcement at the base was selected to resist this moment in combination with gravity loads for appropriate load combinations (for example, ACI 318-99 Section 9.2). Selection of tension reinforcement at the wall boundary was controlled by the load case with minimum gravity load; that is, $U = 0.9D \pm 1.0E$ (note that the $1.3 \times 1.1E = 1.43E$ of ACI was replaced by 1.0E because the UBC-97 loads were already factored, and that the 1.1 multiplier in the exception

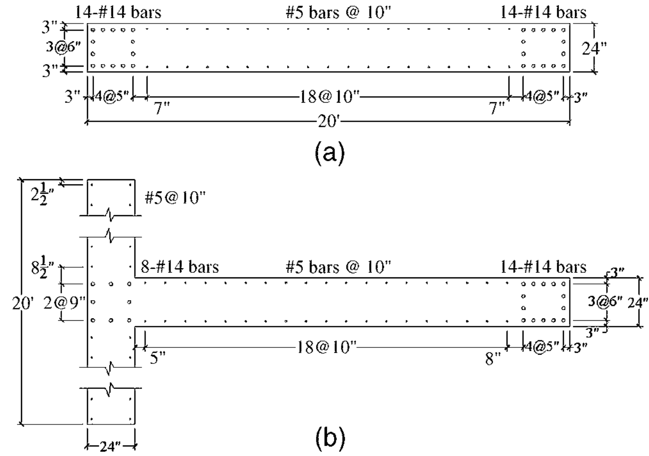

Fig. 3—Preliminary wall designs: (a) Wall R; and (b) Wall T.

of UBC-97 Section 1612.2.1 was not applied). The tension force at the wall boundary region was approximated as $T_u = (M_u - 0.9 \times DL \times 0.4l_w)/0.8l_w$ = 2370 kips (10.54 MN) (Fig. 2), where l_w was the wall length (20 ft [6.1 m] in this case). The required area of tension steel was $A_s = T_u/\phi f_y$ = 2370 kips/(0.9 × 60 ksi) = 44 in.^{2} (28,300 mm^2). Two curtains of shear reinforcement (No. 5 at 10 in.; 250 mm) were used for the 2 ft-thick (0.61 m) web to provide adequate shear strength (ϕVn = 1450 kips [6450 kN] > Vu = 1000 kips [4450 kN]).

For Wall R, 18 No. 14 bars were selected for boundary longitudinal reinforcement (A_b = 2.25 in^2 [1452 mm^2]). For Wall T (wall with T-shaped cross section), the boundary longitudinal reinforcement at the web-flange intersection was reduced by the amount of distributed flange reinforcement within the effective flange width. In this case, the entire flange was assumed to be effective because an effective flange width of 9 ft (15% of the wall height) on each side of the wall web was less than the 25% limit specified in ACI 318-99 Section 21.6.5.2 (Wallace 1996). Assuming that the selection of flange web reinforcement was controlled by minimum requirements, two curtains of web horizontal and vertical reinforcement were used for the 2 ft-thick (0.61 m) flange (No. 5 at 10 in., or ρ = 0.0026). The area of boundary longitudinal reinforcement at the web-flange intersection was reduced by 13.64 in^2 to account for the contribution of the 22 pairs of No. 5 flange vertical bars to tension reinforcement at the wall boundary. Wall strength was verified by using a strain compatibility analysis to generate a ϕP_n-ϕM_n diagram for an extreme fiber compression strain of 0.003. The preliminary design was slightly conservative, and the boundary longitudinal reinforcement was reduced from 18 to 14 No. 14 vertical bars. The preliminary wall designs are shown in Fig. 3. The design load cases plot inside the safe region of ϕP_n-ϕM_n diagrams for each wall; therefore, the designs are acceptable.

At this point, the walls have been designed to have adequate design strength for P-M interaction and for shear. In the following sections, detailing requirements and their impact on wall design and behavior are investigated for ACI 318-95 and ACI 318-99.

ACI 318-95 provisions

The need for special transverse reinforcement at the boundaries of structural walls was evaluated using a stress-based index. This evaluation was accomplished using a linear elastic analysis with element stiffness values based on gross concrete cross-sectional dimensions. For the two cases considered, the extreme fiber compression stresses were computed as 2.47 ksi (17.0 MPa) for Wall R (symmetric), and 0.70 and 1.60 ksi (4.8 and 11.0 MPa) for Wall T, with the flange in compression and tension, respectively. The limiting stress was $0.2f_c' = 1.0$ ksi (6.9 MPa).

For most wall configurations, the extreme fiber compressive stress exceeded the limit of $0.2f_c'$, and specially detailed boundary columns were required (Wallace and Moehle 1992). This was evident from the evaluation of Case R, where the calculated stress was 2.5 times the stress limit; therefore, well-detailed boundary elements are required at both ends of the wall. For Case T, the stress limit was exceeded at the free end of the web (1.6 versus 1.0 ksi), but not at the web-flange intersection (0.7 versus 1.0 ksi). Therefore, a well-detailed boundary element is required at the free end of the web, but not at the web-flange intersection. Where the $0.2f_c'$ limit is exceeded, the contribution of the web concrete and reinforcement are neglected, and the boundary element is required to resist the design loads.

The well-detailed boundary column was proportioned as a short column in compression to resist the overturning moment in addition to the entire axial load (ACI 318-95, Section 21.6.6.1). The size of the boundary column is determined as

$$M_u/(l_w - h_c) + P_u = 0.8\phi[(0.85f_c'(A_g - A_s) + A_s f_y)] \quad (1)$$

where

M_u and P_u = wall overturning moment and wall axial load that result in the largest boundary column, respectively;

l_w = length of wall in direction of applied lateral load;

h_c = boundary column dimension in direction of applied lateral load;

A_g = gross boundary column area; and

A_s and f_y = area and yield stress of longitudinal boundary reinforcement, respectively.

For Walls R and T, M_u = 44,000 ft-kips (59.6 MN-m); P_u = 1000 kips (4480 kN); ϕ = 0.7; A_s = 40.5 in^2 (26,130 mm^2); f_y = 60 ksi (414 MPa); and h_c = 32 in. (813 mm) is assumed; therefore, $A_g \geq 958$ in^2 (0.62 m^2) or a 31 in.-square (787 mm) boundary column. A 32 in.-square (813 mm) boundary column was used. Transverse reinforcement for the boundary column was selected using Section 21.4.4 of ACI 318-95.

The web of the wall was then designed to resist the design shear force using Eq. (21-7) of ACI 318-95. Typically, sufficient shear strength can be attained with minimum web reinforcement (ρ_{min} = 0.0025) and a relatively narrow web. For both walls, the design shear was V_u = 1000 kips (4448 kN) and an 18 in.-thick (457 mm) web with two curtains of No. 5 at 12 in. (305 mm) provided sufficient shear strength (ϕVn = 1150 kips [5115 kN]). The final wall designs are shown in Fig. 4.

The special transverse reinforcement is required until the extreme fiber compressive stress computed using $P/A \pm Mc/I$ is less than $0.15f_c'$. Assuming the wall cross sections do not change with height, special transverse reinforcement may be terminated between the third and fourth floors for Wall R, where the computed stresses were $0.24f_c'$ and $0.13f_c'$, respectively. Therefore, the special transverse reinforcement would extend to the bottom of the fourth floor, or 36 ft (11.0 m) from the base of the wall, for Wall R.

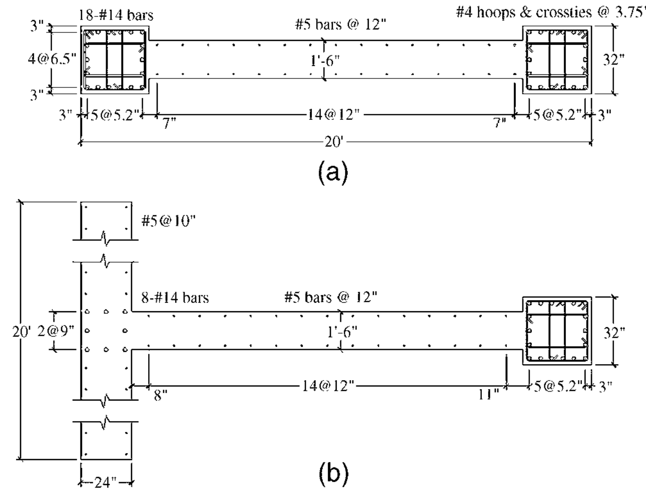

Fig. 4—Final wall designs: (a) Wall R; and (b) Wall T.

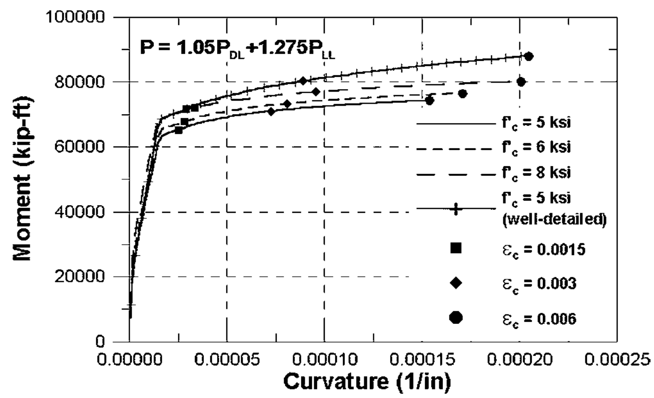

Fig. 5—Moment-curvature comparisons.

Wallace and Moehle (1992) showed that the stress level of $0.2f_c'$ is likely to be exceeded in essentially all reasonably configured wall buildings (with f_y = 60 ksi (414 MPa) and f_c' = 4 ksi (27.6 MPa). In their analysis, the wall area to floor area ratio (the sum of the wall webs aligned in the direction under consideration, or 0.0053 for the preliminary design with a 2 x 20 ft [0.61 x 6.1 m] wall) exceeding approximately 0.15 was needed to avoid the use of well-detailed regions at wall boundaries. The number of walls needed to achieve this ratio is impractical. In addition, it is uneconomical to provide the substantial quantities of transverse reinforcement required over more than 1/2 the building height for a significant number of walls in a building. Therefore, a design with ACI 318-95 generally results in relatively few, well-detailed, walls (that is, buildings with low redundancy).

Given the format of ACI 318-95, increasing the wall cross-sectional dimensions and using higher-strength concrete are potential ways to avoid using specially detailed boundary elements (versus a large wall area to floor area ratio). For example, if the dimensions of Wall R are increased to 2 x 30 ft (0.61 x 9.1 m), the use of f_c' = 6 ksi (41.4 MPa) avoids the need for well-detailed boundary elements ($P_u = 1.05P_{DL} + 1.275P_{LL}$ = 1000 kips (4457 kN); M_u = 44,000 ft-kips (59.6 MN-m); $f_c = P_u/A + M_u c/I = 1.13$ ksi $< 0.2f_c' = 1.2$ ksi [8.3 MPa]). Although the use of a longer wall section is desirable, architectural constraints commonly limit this option; therefore, the use of higher-strength concrete is the more likely option. The use of higher-strength concrete to avoid the use of well-detailed boundary elements does not necessarily result in a better design. The moment-curvature relations plotted in Fig. 5 for a 2 x 20 ft (0.61 x 6.1 m) wall cross section with f_c' values of 5, 6, and 8 ksi (27.6, 41.4, and 55.2 MPa) are considered. The concrete in compression was modeled using the stress-strain relation proposed by Saatcioglu and Razvi (1992) with a strain of 0.002 at the peak stress f_c' and a linear descending branch beyond peak stress with the slope defined using a strain of 0.0035 for a stress of $0.85f_c'$. The original wall design, a 2 x 20 ft (0.61 x 6.1 m) wall cross section without special transverse reinforcement, had adequate flexural strength and substantial curvature ductility capacity. The plots revealed that increasing the concrete compressive strength has little impact on strength and curvature ductility relative to the wall with 5 ksi (41.4 MPa) concrete. In general, use of slender high-strength concrete walls is not advantageous (Wallace 1998).

The moment-curvature relations for the rectangular wall sections plotted in Fig. 5 illustrate the primary disadvantage of using a stress-based approach to evaluate detailing requirements at wall boundaries. Over a broad range of extreme fiber-compression strains—approximately 0.0015 to 0.006 for all three walls—the moment does not vary significantly (and thus, the stress does not vary). Therefore, a stress index based on a linear analysis as used in ACI 318-95 cannot distinguish between cases where low or high compressive strains are expected, and thus cannot be expected to provide a rational means to assess wall detailing requirements.

The moment-curvature relation for the well-detailed wall with 32 in.-square (813 mm) boundary columns and an 18 in.-thick (457 mm) web is also plotted in Fig. 5. The ACI 318-95 wall design has substantial curvature ductility capacity as well as a large flexural overstrength, defined herein as the ratio of M_{pr}/M_u (where M_{pr} is the moment strength for an extreme fiber compression strain of 0.003, accounting for actual or probable material properties). The large flexural overstrength has important implications for the design of the foundation and the wall in shear. The foundation supporting the wall should be designed to accommodate this overstrength, otherwise the foundation system will not perform as intended (rocking may occur).

The implications on wall design for shear are best understood by considering the shear that must develop in the wall, given the wall flexural overstrength compared with the wall shear capacity required by design. Assuming the resultant lateral force at peak shear is located at an effective height equal to 1/2 of the wall height from the base (Paulay 1986; Wallace 1994), a shear force of 2680 kips must be resisted to reach the wall flexural strength (for $\varepsilon_c = 0.003$; $V_{u,\,0.003}$ = 80,340 ft-kips/30 ft = 2680 kips). This shear force exceeds the nominal shear capacity of the wall (ϕVn = 1150 kips [5115 kN]) by a significant margin. Even if some level of material overstrength is considered in calculating the wall shear strength (for example, f_c' = 6 ksi [41.4 MPa], and f_y = 75 ksi [517 MPa]), the modified wall shear strength of ϕVn = 1360 kips (6050 kN) is still only approximately 1/2 the shear force required to reach the probable flexural strength of the wall. Therefore, even though considerable detailing is provided at the wall boundaries to ensure ductile flexural response, the wall will likely experience significant shear distress prior to the yielding of boundary flexural reinforcement (for example, crushing in the web has been observed in some tests [Bertero et al. 1984]), and the wall will not perform as implied by the design process.

In the preceding sections, a simple building configuration was used to determine wall flexural and shear strength requirements. Wall detailing requirements were then assessed using the ACI 318-95 requirements. The presentation was intended to depict the design process as well as identify significant drawbacks associated with the process. Significant observations include:

1. The stress limit of $0.2f_c'$ is very likely to be exceeded for structural walls of reasonably configured buildings; therefore, well-detailed boundary regions are likely to be required and to extend over a significant height of the wall;

2. The use of a stress limit as an index to assess the need for well-detailed boundary regions cannot distinguish between cases where low or high levels of compressive strains are expected;

3. The use of higher-strength concrete to avoid the use of well-detailed boundary regions does not result in wall sections with significantly improved deformation capacity;

4. Because of the substantial detailing requirements, the provisions promote the use of relatively few well-detailed walls (low redundancy);

5. Walls requiring well-detailed boundary regions are likely to have substantial flexural overstrength, potentially leading to premature shear distress or unintended poor foundation performance; and

6. Although not specifically discussed, ACI 318-95 does not include provisions for determining the effective flange width, which is a critical design issue.

ACI 318-99 PROVISIONS

The ACI 318-99 provisions address both proportioning (strength) and detailing issues summarized in the preceding discussion. The steps taken to address proportioning issues include defining an effective flange width, the use of a strain-compatibility analysis and equilibrium to determine section flexural strength, and the introduction of capacity design. Two approaches are included to evaluate detailing requirements at wall boundaries, displacement-based and stress based. Background and simple illustrative examples for the 318-99 provisions are provided in the following sections.

Design strength (proportioning)

Requirements for design flexural and axial load strength are contained in Section 21.6.5.1. A strain-compatibility analysis assuming plane sections (linear strain) is used to assess flexural strength; therefore, a *P-M* interaction diagram is used to determine axial load-versus-moment strength requirements. The linear strain assumption is reasonable for slender walls, even for the relatively deep sections commonly used (Thomsen and Wallace 1995; Wallace 1996; Taylor et al. 1996).

Where walls intersect to form L-, T-, C-, or other cross-sectional shapes, the influence of the flange on the behavior of the wall must be considered by selecting appropriate effective flange widths. The effective flange width for a structural wall depends on the imposed level of deformation, as well as whether the flange is in compression or tension (Thomsen and Wallace 1995; Wallace 1996). Section 21.6.5.2 of ACI 318-99 incorporates a relatively simple approach, where the effective flange width is assumed to extend from the face of the web a distance equal to the smaller of 1/2 the distance to an adjacent wall web and 25% of the total wall height. The use of 25% of the total wall height is a compromise value, and is intended to provide a reasonable value for maximum drift levels expected for structural wall buildings (Wallace and Moehle 1992). A single, effective flange width,

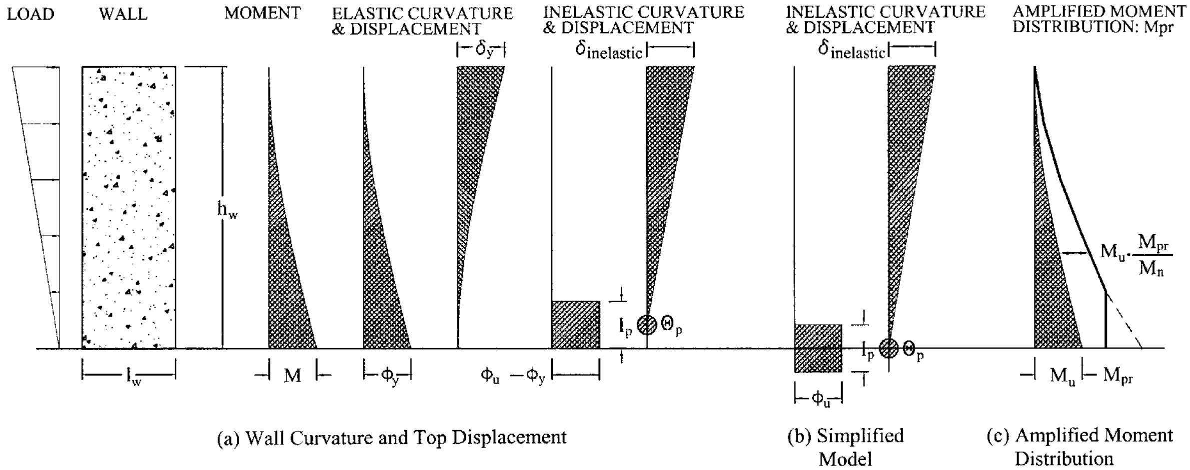

Fig. 6—(a) Wall curvature, top displacement, and amplified moment distribution along wall height.

appropriate for use for the flange in compression or tension, is specified based on recommendations by Wallace (1996).

Prior to assessing detailing requirements, except for the definition of an effective flange width, there are no differences between ACI 318-95 and ACI 318-99. Because an effective flange width was assumed in the ACI 318-95 design of the wall cross sections presented previously, the application of ACI 318-99 does not lead to any differences at this stage of the design. Shear strength requirements for structural walls are covered in Section 21.6.4 and are essentially unchanged from those in ACI 318-95. This topic will be addressed in greater detail in the following sections of this paper.

Detailing requirements

A major change from ACI 318-95 involves the treatment of web concrete and reinforcement. In ACI 318-95, where special detailing is required, the boundary elements are required to resist the entire overturning moment and axial load (that is, the web contribution was neglected), whereas the web concrete and reinforcement are considered in ACI 318-99. Therefore, walls designed using ACI 318-99 will have less flexural overstrength compared with walls designed using ACI 318-95. This topic is also addressed in the following paragraphs, which discuss detailing requirements.

As noted previously, two approaches are included to assess detailing requirements. The first approach involves the use of displacement-based design and is limited to walls controlled by flexure with a single, critical section over the wall height. The second approach is similar in format to that used in ACI 318-95, and applies to all wall configurations.

Displacement-based approach—Displacement-based design of structural walls was introduced in the late 1980s and early 1990s (Moehle and Wallace 1989; Wallace and Moehle 1992) as an alternative to the prescriptive detailing provisions used in the buildings codes at that time (for example, ACI 318-89, UBC-88). A detailed description of the approach for structural walls is presented by Wallace (1994), with additional details provided by Wallace (1995) and Wallace and Thomsen (1995). Experimental verification of the procedure is presented by Taylor et al. (1996). A brief summary of the fundamental concepts is included herein to allow comparisons with the ACI 318-99 code provisions.

The displacement at the top of the wall is computed based on the stiffness of the structural system and the representation of the ground motions used for design. The example building configuration described in Fig. 1, where the lateral stiffness was provided mainly by the structural wall should be considered. Flexural stiffness values over the height of the wall can be determined based on an assumed moment distribution. If the wall reaches the nominal moment capacity at the base for the design loads, then cracking is expected to occur from the base to a height of 37.7 ft (11.5 m), or approximately over the first three floor levels (using a concrete rupture strength equal to $7.5\sqrt{f_c'}$ psi [$0.63\sqrt{f_c'}$ MPa]). Flexural stiffness values of $0.5EI_g$ and $0.8EI_g$ are selected for the cracked and uncracked sections, respectively, based on the values recommended by FEMA 273, Table 6.3 (Federal Emergency Management Agency 1997).

For demonstration purposes, the design displacement for the five-story example buildings were obtained for the equivalent static lateral loads applied to a stick model of Walls R and T. Interaction with the frame elements was not considered, as this level of detail is not necessary for this study.

The displacement at the top of the Wall R was computed to be 5.4 in (137 mm) for a wall flexural stiffness of $0.5EI_g$ over the bottom three floors, and $0.8EI_g$ for the top two levels ($\delta_u \equiv \Delta_M = 0.7R\Delta_S = 0.7 \times 5.5 \times 1.4$ in. = 5.4 in. [137 mm], where Δ_S is the top displacement for the design loads reduced by R). The 0.5 multiplier on EI_g accounts for the reduction in flexural stiffness due to concrete cracking, whereas the 0.8 multiplier accounts for the influence of microlevel cracking at moments less than the cracking moment (for example, those due to temperature and shrinkage). The value of $0.8EI_g$ used for the uncracked portion of the wall was used based on the recommendation of FEMA 273, Table 6.3 (Federal Emergency Management Agency 1997). If a uniform stiffness of $0.5EI_g$ is used over the full height of the wall, the top displacement is 5.5 in. (140 mm).

Use of $0.5EI_g$ for a cracked Wall T may be inappropriate given the influence of the flange. Using a moment-curvature analysis, an effective (cracked) flexural stiffness for the T-wall was found to be $0.30EI_g$ and $0.28EI_g$ for the flange in compression and tension, respectively (the effective EI is the slope of the M-ϕ diagram). For analysis with the flange in tension, a stiffness of $0.3EI_g$ was used for the first level (the cracking moment was exceeded only in the first floor level), whereas a stiffness of $0.8EI_g$ was used for the upper four levels. For analysis with the flange in compression, a stiffness of $0.3EI_g$ was used for the bottom two levels, whereas a stiffness of $0.8EI_g$ was used for the upper three levels. The resulting design displacements are 4.1 and 3.0 in. (104 and 76 mm), for forces resulting in flange in compression and tension, respectively.

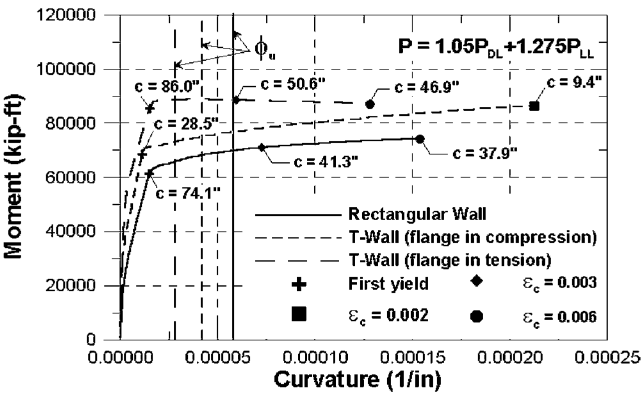

Fig. 7—Moment-curvature comparisons.

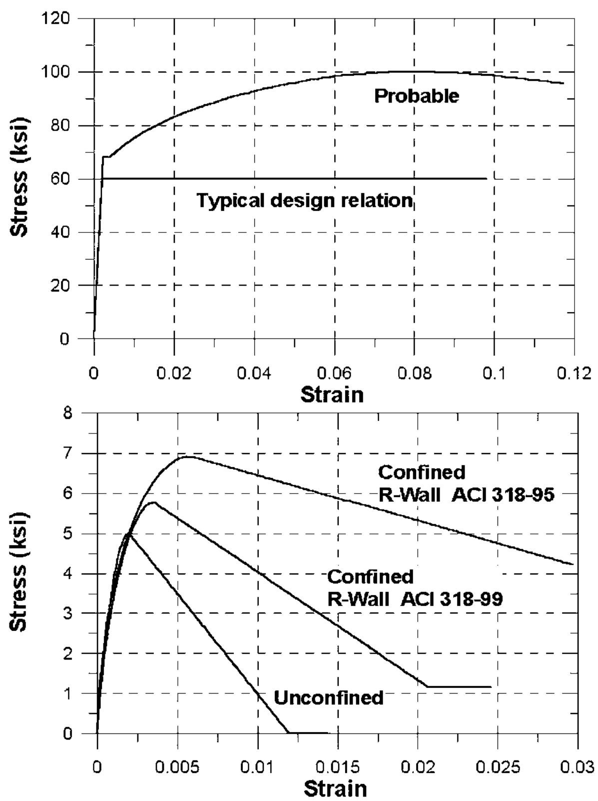

Fig. 8—Material stress-strain relations: (a) steel; and (b) concrete in compression.

The relationship between the wall top displacement and wall curvature for a uniform wall cross section with a single critical section at the base is presented in Fig. 6. For an inverted triangular lateral load, the displacement at the top of the wall can be represented as

$$\delta_u = \frac{11}{40}\phi_y h_w^2 + (\phi_u - \phi_y) l_p \left(h_w - \frac{l_p}{2}\right) \tag{2}$$

The term ϕ_u in Eq. (2) is typically computed for a given design displacement δ_u, yield curvature estimate ($\phi_y = 0.003/l_w$), and plastic hinge length ($l_p = l_w/2$). Moment-curvature diagrams are plotted for Walls R and T with the flange in compression and tension in Fig. 7. Material relations used for the M-ϕ analysis are indicated in Fig. 8 (for probable steel and including confinement). Curvature values for the first yield of reinforcement and an extreme fiber compression strain of 0.003 are noted in

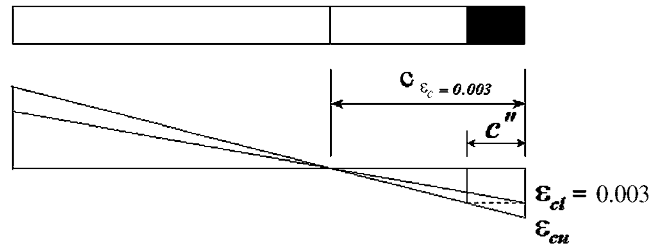

Fig. 9—Wall strain distributions and required length of confinement.

Fig. 7. Ultimate curvatures (ϕ_u) computed using Eq. (2) are noted on Fig. 7 for the design displacements.

The information contained in Fig. 7 reveals that special detailing is not required for Wall R because the ultimate curvature for the design displacement ϕ_u is less than the curvature developed for an extreme fiber strain of 0.003 (that is, the extreme fiber compression strain to achieve the curvature ϕ_u is less than 0.003). For Wall T, special detailing is not required at the boundary where the web and the flange intersect (note that for an extreme fiber compression strain of 0.002, the wall-curvature capacity greatly exceeds the curvature demand), or at the free end of the web. As will be discussed further in the paper, ACI 318-99 requires moderate detailing at the wall boundary to ensure that the longitudinal reinforcement will not buckle prematurely.

If special detailing is required, the length over which it must be provided is computed assuming a linear strain profile over the wall length, with the neutral axis defined using a limiting compression strain (Wallace 1994). A limiting strain between 0.003 and 0.005 is commonly used; a value of 0.003 is used in ACI 318-99. This approach is based on the assumption that the neutral axis does not change significantly within the range of extreme fiber compression strain values typically computed for structural walls. Experimental studies indicate that this assumption is reasonable (Thomsen and Wallace 1995), and the additional work associated with computing the neutral axis depth for each ultimate curvature value computed is not warranted. Figure 9 presents the strain distributions for extreme fiber strains of 0.003 and ε_{cu}, as well as the wall length over which special detailing is required. The quantity and distribution of transverse reinforcement required at the wall boundary is discussed later.

The information presented in the preceding paragraphs was necessary to provide sufficient background to understand the basis for the ACI 318-99 provisions. The ACI 318-99 provisions are based on a simplified version of the material presented in the preceding paragraphs. The model of Fig. 6(a) is simplified as shown in Fig. 6(b). The model shown in Fig. 6(b) neglects the contribution of elastic deformations to the top displacement, and moves the centroid of the plastic deformations to the base of the wall. This model has been shown to work well over the range of parameters of interest (Moehle 1992). Based on the model of Fig. 6(b), the relationship between the top displacement and the curvature at the base of the wall is

$$\delta_u = \theta_p h_w = (\phi_u l_p) h_w = \left(\frac{\varepsilon_{cu}}{c}\frac{l_w}{2}\right) h_w \tag{3}$$

$$\varepsilon_{cu} = 2\left[\frac{\delta_u}{h_w}\right]\left[\frac{c}{l_w}\right]$$

where

c = neutral axis depth computed for $\varepsilon_c = 0.003$;
δ_u = design displacement;
h_w = wall height;
l_w = wall length;
ϕ_u = ultimate curvature;
θ_p = plastic rotation at the base of the wall; and
ε_{cu} = extreme fiber compression strain associated with the ultimate curvature ϕ_u.

The wall length over which the strains exceed a limiting value (for example, $\varepsilon_{cl} = 0.003$), denoted as c'', can be determined using similar triangles (Fig. 9) as

$$c'' = c\left[1 - \frac{\varepsilon_{cl}}{\varepsilon_{cu}}\right] \tag{4}$$

A relation for the required length of confinement can be developed by combining Eq. (3) and (4) as

$$\frac{c''}{l_w} = \frac{c}{l_w} - \frac{\varepsilon_{cu}/2}{(\delta_u/h_w)} \tag{5}$$

The term c/l_w in Eq. (5) accounts for the influence of material relations (for example, f_c' and f_y), axial load, geometry, and quantities and distribution of reinforcement, whereas the term $(\varepsilon_{cu}/2)/(\delta_u/h_w)$ accounts for the influence of system response (roof displacement) and the limiting concrete strain (taken as 0.003 in ACI 318-99).

The 318-99 provisions, presented in Section 21.6.6.2, were directly derived from the information presented in the preceding paragraphs. From Eq. (3), special detailing is required if

$$c \geq \frac{\varepsilon_{cl}}{2}\frac{l_w}{(\delta_u/h_w)} = \frac{0.003}{2}\frac{l_w}{(\delta_u/h_w)} = \tag{6}$$

$$\frac{l_w}{667(\delta_u/h_w)} \approx \frac{l_w}{600(\delta_u/h_w)}$$

Therefore, if the neutral axis for a given wall exceeds this limit, it implies that the extreme fiber compression strain exceeds the limiting strain of 0.003, and special detailing must be provided. In ACI 318-99, the term δ_u/h_w must be equal to or exceed 0.007. This lower limit on the mean drift ratio is included to ensure that all walls controlled by flexure have modest deformation capacities, as well as to guard against modeling errors that might underestimate the design displacement. The substitution of $\delta_u/h_w = 0.007$ into Eq. (6) requires that special detailing be provided where $c \geq 0.24l_w$.

Where special detailing is required at the wall boundary, it must be extended vertically a distance not less than the larger of l_w and $M_u/4V_u$ from the critical section. The lengths specified are intended to be an upper-bound estimate of the plastic hinge length for structural walls (Paulay 1986; Wallace 1994).

The displacement-based approach presented is founded on the concept that yielding is limited to a single critical section, usually at the base of the wall. To ensure that this mechanism controls the response of the wall, adequate flexural strength must be provided over the wall height to force yielding to take place at the assumed critical section. This issue is addressed briefly in the commentary to Section 21.6.6.2. Additional details are available in the papers authored by Paulay (1986) and Wallace (1994).

One approach to satisfying this criterion is to amplify the moment capacity over the wall height to promote yielding at the critical section (Fig. 6(c)). In this approach, the probable moment capacity at the critical section is calculated, and the design moments at other locations are amplified by the ratio of the probable moment to the nominal moment at the critical section M_{pr}/M_n to determine where reinforcement may be reduced over the wall height. For a wall with a constant cross section, it is not the intent of this provision to require more reinforcement over the wall height than is required at the critical section, but to limit where reinforcement is terminated.

Modified stress-based approach—A stress-based approach was included in ACI 318-99, Section 21.6.6.3, to address wall configurations where the application of displacement-based design is not appropriate (for example, some perforated walls, walls with setbacks, walls not controlled by flexure [Maffei et al. 2000]). Maintaining the stress-based approach also provides continuity between the ACI 318-95 and ACI 318-99 code; however, modifications were incorporated to address shortcomings of the ACI 318-95 provisions summarized previously.

The stress limit where special detailing is required at the wall boundaries is unchanged from that in ACI 318-95 ($0.2f_c'$); therefore, the calculations presented previously are valid, and special detailing is required at both ends of Wall R and at the free end of the web for Wall T. The special detailing must be extended over the height of the wall from the critical section until the calculated stress is $0.15f_c'$ (the same as ACI 318-95).

A major difference between ACI 318-99 and ACI 318-95 involves flexural strength requirements for walls where specially detailed boundary elements are required. For ACI 318-95, the boundary columns are required to resist the compression due to the earthquake overturning effect, as well as the full axial load as a short column. The application of this requirement typically results in a substantial increase in wall flexural strength, producing deleterious effects for wall shear and foundation forces, as noted previously. No such requirement exists in ACI 318-99. Moment-curvature relationships are compared later for Wall R to assess the impact that this change has on the flexural strength.

Requirements for transverse reinforcement—The background on the quantity of special transverse reinforcement required at the wall boundary, and the wall length over which special detailing must be provided, is covered in Section 21.6.6.4 of ACI 318-99. The provisions of this section apply to both the displacement-based evaluation (Section 21.6.6.2) and the stress-based evaluation (Section 21.6.6.3).

The wall length over which special transverse reinforcement must be provided is based on Eq. (5), with a value of $\delta_u/h_w = 0.015$

$$\frac{c''}{l_w} = \frac{c}{l_w} - \frac{1}{667(\delta_u/h_w = 0.015)} = \frac{c}{l_w} - 0.1 \geq \frac{c}{2} \tag{7}$$

The value of δ_u/h_w of 0.015 was selected to provide an upper-bound estimate of the mean drift ratio for typical structural wall buildings constructed in the U.S. (Wallace and Moehle 1992); therefore, the length of the wall that must be confined tends towards the conservative side for many buildings. The value of $c/2$ represents a minimum length of confinement.

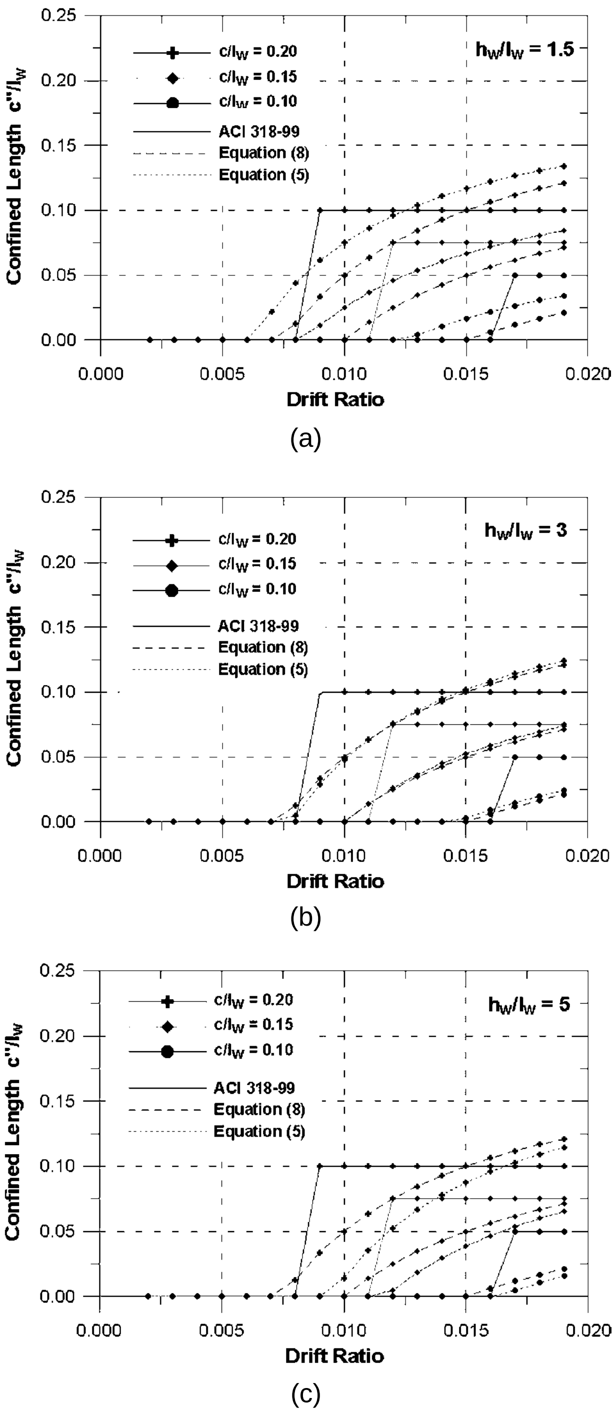

Fig. 10—Confinement length-versus-drift ratio relations: (a) for aspect ratio of 1.5; (b) for aspect ratio of 3; and (c) for aspect ratio of 5.

The quantity of transverse reinforcement required at the wall boundary is selected to satisfy ACI 318-99 Eq. (21-4). ACI 318-99 Eq. (21-3) is not referenced because it is based on maintaining the pre- and post-spalling capacity of a column subjected to pure compression, which does not apply to walls.

Displacement-based design (ACI 318-99)—In the development of code provisions, it is necessary to achieve a balance among accuracy, ease of application, and practicality, as well as to consider potential sources of uncertainty. Figure 10 provides a comparison of results obtained using the more-detailed representation of displacement-based design embodied in Fig. 6(a), by combining Eq. (2) and (5)

$$\frac{c''}{l_w} = \frac{c}{l_w} - \frac{(\varepsilon_{cu}/2)(1 - 1/(4A_w))}{\left(\frac{\delta_u}{h_w}\right)\left(\frac{\varepsilon_{sy}}{1 - c/l_w}\right)\left(\frac{11}{40}A_w - \frac{1}{2} + \frac{1}{8A_w}\right)} \qquad (8)$$

where A_w is the wall aspect ratio (h_w/l_w) and ε_{sy} is the yield strain of the tension steel (used to estimate yield curvature). Equation (8) is compared with the simplified representation embodied in Fig. 6(b) and Eq. (5).Three cases are considered to compare these relations for a range of variables. The plots reveal that the simple representation of Fig. 6(b) represents well the more-detailed representation of Fig. 6(a). The relations are nearly the same for an aspect ratio of 3, and very close for aspect ratios of 1.5 and 5. A third relation, which represents the ACI 318-99 provisions—that is, Eq. (6) and (7)—is also plotted in Fig. 10. The ACI 318-99 relations tend to be conservative for mean drift ratios less than 0.015, and slightly unconservative for higher mean drift ratios (where the minimum length of confinement is not controlled by $c/2$). This is expected, given the use of 0.015 as a reasonable upperbound on mean drift ratio in Eq. (7). Overall, the ACI 318-99 relation is easy to use and reasonably accurate.

Sources of uncertainty that should be considered when applying the ACI 318-99 provisions include the modeling assumptions used to predict the design displacement, the assumed plastic hinge length at the critical section ($l_p = 0.5l_w$), and the actual material properties. With the background information presented, the influence of these parameters on the wall design can be considered. The greatest uncertainty is likely to be associated with the design-basis earthquake (that is, the earthquake ground motions used for design), where significant variations from the mean spectra used for design are observed (for example, Dobry et al. 2000). A comprehensive study on the variability of spectral ordinates for various geologic classification schemes is presented by Stewart, Liu, and Baturay (2001). Use of the displacement-based design approach in ACI 318-99 implies that the design can be manipulated to achieve a specified level of performance (for example, no spalling); however, it is important to understand that the actual performance may vary significantly given the uncertainties noted, particularly those associated with the design-basis earthquake.

Application of ACI 318-99

The application of the displacement-based design provisions for structural walls in ACI 318-99 for Walls R and T is summarized in the following paragraphs.

Wall R—Flexural and shear strength requirements for Wall R are the same as for ACI 318-95; therefore, the wall cross section (at the base of the wall) is the same as that given in Fig. 3. The design displacement, determined using stiffness values of $0.5I_g$ over the bottom three floors and $0.8I_g$ for the top two levels, is 5.4 in. (137 mm) for an equivalent static lateral load analysis (story forces used for design). According to UBC-97, the design displacement, $\Delta_m = 0.7R\Delta_s$, where R is the force-reduction factor (5.5), and Δ_s is the displacement for the code level (design) forces (that is, the forces have been reduced by R to determine the design moments, shears, and axial loads on the structural members). In this presentation, Δ_m and δ_u are equivalent quantities (for more discussion on this topic, refer to Wallace [1996]). The term $\delta_u/h_w = 0.0075$,

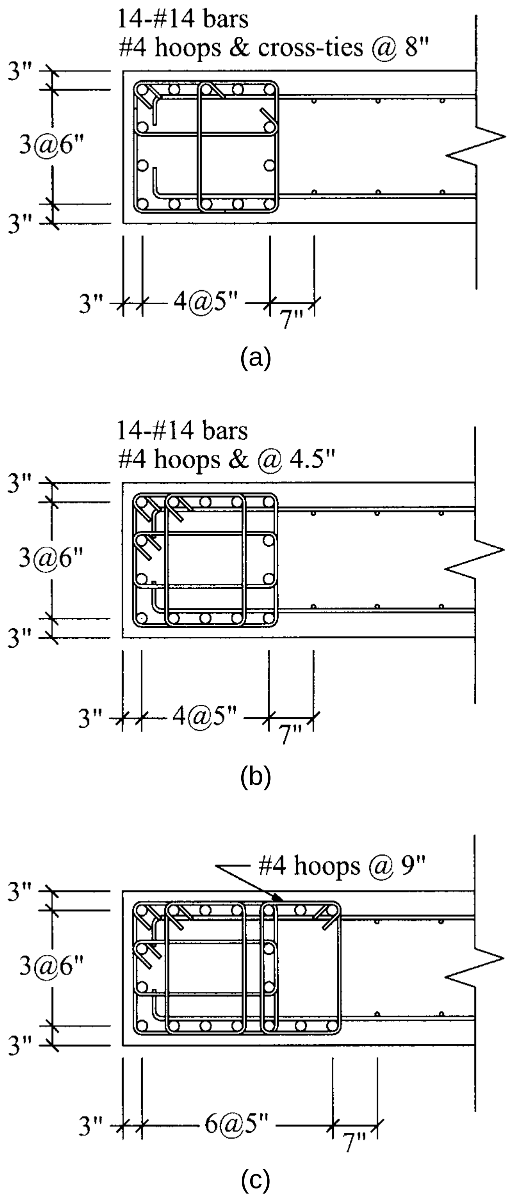

Fig. 11—Detail for boundary region of Wall R (ACI 318-99): (a) final design; (b) specially detailed, enclosing all boundary longitudinal reinforcement; and (c) specially detailed, not enclosing all boundary longitudinal reinforcement.

which exceeds the minimum value of 0.007; therefore, a value of 0.0075 is used in Eq. (10).

Using a strain compatibility analysis for $\varepsilon_c = 0.003$ and the material relations of Fig. 8 (for design, a Whitney Stress Block and a design relation of the reinforcement stress-strain relation would likely be used), the maximum neutral axis depth for Wall R is 41.3 in. (1.05 m) for load case Eq. (9-2) of ACI 318-99: $U = 1.05D + 1.275L + 1.0E$ (the load factor on E is changed from 1.4 to 1.0 to account for the fact that UBC-97 earthquake loads presented previously have already been factored). Substitution into Eq. (6) results in

$$c \geq \frac{(l_w = 240\text{ in.})}{600(0.0075)} = 53.3\text{ in.} \tag{9}$$

Because the neutral axis depth of 41.3 in. (1.05 m) to reach an extreme fiber compression strain of 0.003 is less than

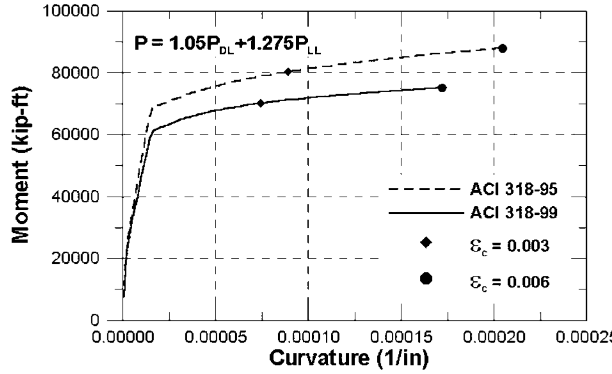

Fig. 12—Influence of detailing requirements on flexural strength.

53.3 in. (1.35 m), no special transverse reinforcement is needed. Because the amount of reinforcement at the wall boundary is substantial (14 No. 14 bars), however, moderate transverse reinforcement is required according to Section 21.6.6.5(a). Ties (No. 4) are provided at 8 in. (203 mm) on center over a wall length equal to the greater of $c - 0.1l_w$ = 17.3 in. (439 mm) and $c/2$ = 20.7 in. (526 mm). The final detail for the boundary region is shown in Fig. 11(a).

For comparison, moment-curvature analyses of Wall R, designed according to ACI 318-95 and ACI 318-99, are presented in Fig. 12 (again, using the material relations of Fig. 8). For $\varepsilon_c = 0.003$, the wall designed according to ACI 318-99 had a moment strength of $1.6M_u$, whereas the wall designed according to ACI 318-95 had a moment strength of $1.8M_u$ (where M_u is the design moment strength). The ACI 318-99 wall design had less flexural overstrength, and thus, addressed one of the shortcomings of ACI 318-95. It should be noted, however, that the potential for shear distress prior to flexural yielding still exists. For a probable moment strength of $1.6M_u$, and assuming the resultant lateral load acts at 1/2 the wall height, a shear force of 2340 kips is needed to produce the probable moment at the critical section (the base of the wall). The shear for the probable moment is 1.6 times the shear capacity ϕV_n. To ensure that the majority of inelastic response is from flexural yielding, the shear strength of the wall should be increased—that is, capacity design of the wall in shear should be considered. Capacity design of structural walls in shear is not addressed in ACI 318-99, primarily because shear distress of structural walls has not been observed to produce life safety or collapse problems. If the design focus is on performance, however, capacity design of the wall for shear may be appropriate.

Wall T—Similar to Wall R, the flexural and shear strength requirements for Wall T are the same as those in ACI 318-95; therefore, the wall cross section (at the base of the wall) is the same as given in Fig. 3. The design displacements for Wall T were evaluated independently for loads in each direction, due to the variation in flexural stiffness and cracking moments, as noted previously. For the flange in tension, the design displacement was 3.0 in. (76 mm) and $\delta_u/h_w = 0.0042$. For the flange in compression, the design displacement is 4.1 in. (104 mm) and $\delta_u/h_w = 0.0057$. Because the ratio of $\delta_u/h_w < 0.007$ for both cases, a value of 0.007 must be used in Eq. (5) to determine the critical neutral axis depth.

$$c \geq \frac{(l_w = 240\text{ in.})}{600(0.007)} = 57.1\text{ in. (1.45 m)} \tag{10}$$

Therefore, for loads in either direction, if the neutral axis depth exceeds 57.1 in. (1.45 m), special detailing must be provided at the wall boundary. Using a strain compatibility analysis for $\varepsilon_c = 0.003$, the maximum neutral axis depth for Wall T is 50.6 in. (1.29 m) for the flange in tension, and 8.2 in. (0.21 m) for the flange in compression. Because these neutral axis depths do not exceed the limit of 57.1 in. (1.45 m), no special transverse reinforcement is needed. Transverse reinforcement is provided to satisfy Section 21.6.6.5(a) at both wall boundaries.

Wall R (modified)—Because neither Wall R or T required special detailing in the preceding evaluations, the drift ratio δ_u/h_w is arbitrarily increased to 0.012 for Wall R to demonstrate a procedure for sections requiring special detailing. The critical neutral axis depth is

$$c \geq \frac{(l_w = 240 \text{ in.})}{600(0.012)} = 33.3 \text{ in. } (0.85 \text{ m}) \qquad (11)$$

Because the neutral axis depth of 41.3 in. (1.05 m) to reach an extreme fiber compression strain of 0.003 is greater than the critical depth of 33.3 in. (0.85 m), special detailing is required over a depth equal to the greater of $c - 0.1l_w = 17.3$ in. (439 mm) or $c/2 = 20.7$ in. (526 mm). The required area of transverse reinforcement is specified in Eq. (21-4). A plan view of the reinforcing details at the wall boundary region is presented in Fig. 11(b). In this case, a depth of 20.7 in. (526 mm) encompasses nearly all the boundary longitudinal reinforcement; therefore, the transverse reinforcement is provided that encloses all longitudinal reinforcement. The special detailing at the wall boundaries is required to extend a height equal to the greater of $l_w = 20$ ft (6.1 m) and $M_u/4V_u = 44{,}000$ ft-kips/(4 × 1000 kips) = 11 ft (3.35 m), or 20 ft (6.1 m) in this case.

A hypothetical distribution of longitudinal reinforcement, where the special transverse reinforcement does not extend beyond all the longitudinal reinforcement, is shown in Fig. 11(c). ACI 318-99 does not address what, if any, transverse reinforcement is required for the main longitudinal reinforcement outside the specially detailed region. Because this reinforcement is susceptible to buckling, at a minimum, transverse reinforcement satisfying Section 7.10 of ACI 318-99 should be provided as shown in Fig. 11(c).

SUMMARY AND CONCLUSIONS

Major changes to the provisions for proportioning and detailing of structural walls were incorporated into ACI 318-99 to take advantage of displacement-based design, as well as to address shortcomings associated with prior ACI 318 codes. These changes include defining effective flange widths, reducing flexural overstrength, reducing the wall height over which special detailing is required, and providing consistent detailing requirements for stress- and displacement-based evaluations. The changes impact the design of slender walls significantly. Examples were used to identify shortcomings associated with the ACI 318-95 requirements, to explain how the ACI 318-99 requirements address these shortcomings, as well as to clarify the intent of the new provisions.

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