8.

Evaluation of UBC-94 Provisions for Seismic Design of RC Structural Walls

John W. Wallace, M.EERI

New provisions for seismic design of reinforced concrete structural walls were incorporated into the 1994 version of the Uniform Building Code. The new provisions are based on the use of a displacement-based design methodology, which is a significant departure from prior codes. The format of the UBC-94 provisions offer significant advantages over previous design formats, with both simplified and detailed design approaches. As is the case with any significant code change, proper application of the new provisions requires a thorough understanding of the design methodology used to develop the provisions as well as its limitations. Given these needs, an overview of displacement-based and the UBC-1994 requirements is provided. The methods employed in UBC-94 to estimate the maximum displacement response are found to significantly underestimate maximum displacement; therefore, direct application of the provisions could lead to unconservative designs. Recommendations to address this shortcoming, as well as potential to improve the provisions in other areas are presented.

INTRODUCTION

Considerable work in recent years has improved the understanding of the lateral load behavior and design of structural walls. Shortcomings and the inherent conservatism of current design provisions have been identified (Wood, 1991; Wallace and Moehle, 1992), and a displacement-based design methodology has been developed (Moehle and Wallace, 1989; Wallace and Moehle, 1992; Wallace and Thomsen, 1993; 1995; Wallace 1994a,b; 1995). Based on this research, and a review of previous research, the Structural Engineers Association of California (SEAOC) developed new design recommendations for shear wall buildings that have been incorporated into the 1994 UBC. The recommended provisions use a displacement-based design methodology and will result in a relaxation of detailing requirements for most structural walls.

[1]Department of Civil Engineering, P. O. Box 5710, Clarkson University, Potsdam, NY 13699-5710

As with any new code provision, there are many factors that influence the final code document such that the basis for the code changes are sometimes lost in the detail that emerges. However, given the significant departure of the new provisions (UBC, 1994) from previous design practice (UBC, 1991), it is important that detailed background material be presented and that the limitations and shortcomings of the new provisions be discussed. Given these needs, the objectives of this paper are to: (1) provide an overview of displacement-based design for structural walls, (2) review appropriate levels of design displacement, (3) provide an overview of the UBC (1994) provisions, (4) review critical aspects of the proposed UBC (1994) provisions, and (5) provide recommendations for future code development.

DISPLACEMENT-BASED DESIGN OF RC STRUCUTRAL WALLS

The development of displacement-based design procedures for RC structural walls was an extension of analytical studies of buildings following the 1985 Chile earthquake (Wallace and Moehle, 1989). Overall damage to the more than 300 RC structural wall buildings in the Valparaiso and Viña del Mar region of Chile was light despite the long duration of the strong ground motions and the relatively lax detailing requirements for transverse reinforcement at the wall boundaries compared with U.S. practice (Wallace and Moehle, 1989; 1993). Although a number of factors influenced the observed building performance, analytical studies indicated that light wall damage could be traced to the stiffness of the structural system, which limited the deformations imposed on the lateral force resisting system (Wood et al., 1987; Wallace and Moehle, 1989; 1992; 1993; Wood, 1993).

Subsequent studies (Moehle and Wallace, 1989; Wallace and Moehle, 1992; Wallace, 1994a,b; Wallace, 1995; Wallace and Thomsen, 1993; 1995) focused on the development of a displacement-based design procedure for RC structural walls. In a displacement-based format, the lateral displacement pattern imposed on a structural wall is estimated and related to the deformations imposed at the base of the wall (Fig. 1). Using established procedures that account for the distribution of the elastic and inelastic deformations along the wall height, a relationship can be derived between the roof drift and the ultimate curvature imposed as a result of the earthquake ground motions at the base of the wall (Wallace and Moehle, 1992). This relationship can be expressed as Eq. (1a). A simplified version of this relationship, Eq. (1b), which neglects the contribution of the elastic deformations to the roof drift, was proposed by Moehle (1992).

$$\phi_u l_w = 0.0025\left(1 - \frac{1}{2}\frac{h_w}{l_w}\right) + 2\,\frac{\delta_u}{h_w} \qquad \text{(1a)}$$

$$\phi_u l_w = 2\,\frac{\delta_u}{h_w} \qquad \text{(1b)}$$

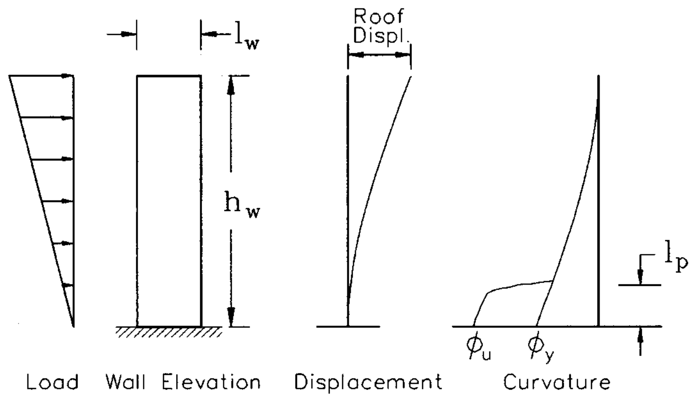

Figure 1 Wall Definitions and Response Characteristics

In Eq. (1), h_w is the wall height, l_w is the wall length, ϕ_u is the ultimate curvature, ε_{cu} is the extreme fiber concrete compression strain, and δ_u is the design roof displacement. Based on the assumption that plane-sections-remain-plane after load application, the maximum compression strain imposed on the wall cross-section is determined by rearranging Eq. (1b). The result is expressed as Eq. (2).

$$\varepsilon_{cu} = 2\,\frac{\delta_u}{h_w}\,\frac{c}{l_w} \tag{2}$$

In Eq. (2), the term δ_u / h_w is the roof drift and c is the depth of the neutral axis from the extreme compression fiber computed based on equilibrium requirements.

The relationship between the roof drift estimate of a building and the resulting wall curvature or normal strains is clearly apparent from Equations (1) and (2); therefore, a critical aspect of using a displacement-based design process involves estimating the roof drift ratio. This topic is discussed in detail in the following paragraphs.

Considerable research has been conducted on displacement response of elastic and inelastic systems (Miranda, 1993a,b; Shibata and Sozen, 1976; Shimazaki and Sozen, 1984; Qi and Moehle, 1991; Wallace and Moehle, 1992; Moehle 1992; Wallace 1994a,b; Bonacci 1994; Seneviratna and Krawinkler, 1994). These works include response studies of: (1) computer models representing single-degree-of-freedom (SDOF) and multi-degree-of-freedom (MDOF) systems, (2) instrumented and uninstrumented buildings subjected to earthquake ground motions, and (3) SDOF and MDOF building models on earthquake simulators. Important aspects of these studies with respect to displacement-based design of RC structural walls is discussed in the following paragraphs.

Miranda (1993a,b) studied the displacement response of SDOF elastic and bilinear systems for a large number of ground motions. For ground motions obtained on rock or alluvium, the maximum displacement response of an inelastic system is approximately equal to the maximum displacement response of an elastic system if the initial (fundamental) period of the building exceeds 0.5 to 0.75 seconds (Fig. 2, for $\mu_\delta < 4$). This is commonly referred to as the ``equal displacement rule", and is often used as a basis for code development. For shorter periods, the maximum displacement response of an inelastic system is greater than that of an elastic system, and depends on the strength and period of the building (Fig. 2). It is noted that the relations plotted in Fig. 2 were computed using a elastic-perfectly plastic force-deformation model; therefore, incorporation of moderate strain hardening would result in lower deformation demands in the short period range.

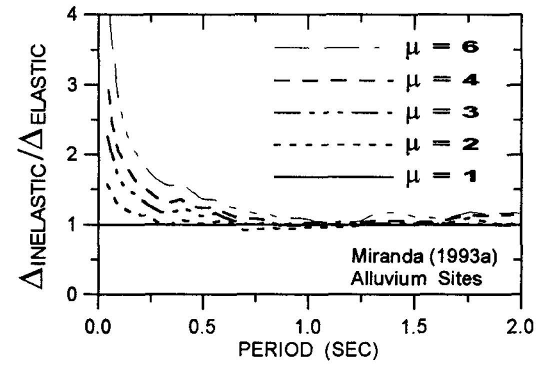

Figure 2 Mean Ratio of Inelastic Displacement Response of SDOF Systems

Similar trends to those reported by Miranda (1993) are reported for analytical studies of real buildings (Wallace and Moehle, 1993), and for SDOF and MDOF systems tested on earthquake simulators (Shimazaki and Sozen, 1984; Bonacci, 1994; Fig. 3). For the earthquake simulator tests, if the initial (fundamental) period of the building model T_0 exceeds the characteristic ground period T_g, then the maximum displacement response for an inelastic system can be reasonably estimated by the maximum displacement response of an elastic system. For initial building period less than T_g, the maximum inelastic displacement response exceeds that for an elastic system (Fig. 3). The characteristic ground period can be estimated as the period where the constant acceleration region and constant velocity region coincide on a 4-way log plot of spectral ordinates, and is approximately 0.5 sec for the results reported in Figure 3.

The studies outlined in the preceding paragraphs indicate that the maximum elastic displacement is a reasonable estimate of maximum displacement response for buildings on rock or alluvium with initial period greater than approximately 0.5 seconds. In addition, for these "long period" structures, the maximum displacement response is relatively insensitive to the strength of the structure. Therefore, the maximum displacement response can be estimated using elastic analysis techniques commonly employed in structural engineering practice provided that the effects of concrete cracking are considered. For shorter period structures, the maximum displacement response is greater than the maximum elastic response, and depends on the strength and period of the structure. To estimate the maximum displacement response for "short period" structures, the maximum elastic displacement should be amplified. The amplification factor can be approximated using the relations presented by Miranda (1993a) based on estimates of the initial period of the structure and the anticipated level of inelastic response (displacement ductility).

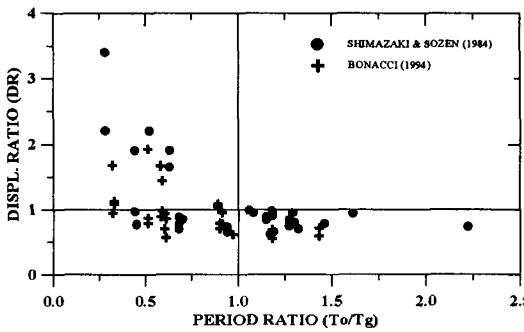

Figure 3 Inelastic Displacement Response Ratios for Earthquake Simulation Studies

A displacement-based design approach is attractive for design office practice because both design forces and displacements can be determined using an elastic response spectrum analyses. Either a site specific or generalized spectrum can be used for the analysis. For proportioning of the structural elements, a response spectrum analysis based on using gross-section stiffness values and an equivalent code spectrum modified to account for inelastic response (reduced by R_w) is commonly used. For evaluating detailing requirements, a response spectrum analysis based on using an effective stiffness and an unreduced spectrum ($R_w = 1$) should be used. Modifications in the short period range should be applied based on the information presented in the preceding paragraph. Response correlation studies of instrumented structural wall buildings in low-to-moderate intensity earthquake ground motions indicate using an effective stiffness for structural walls equal to the cracked-section stiffness, or 40 to 50% of the gross-section stiffness is appropriate (Wallace et al., 1990; Yan and Wallace, 1993). Stiffness modifications for other structural elements (beams, slabs, and columns) should also be evaluated. Once the design (roof) displacement has been estimated, well established techniques can be employed to relate roof drift to the maximum curvature or compression strain imposed on the wall cross-section (Eq. 1). Based on the wall normal strain distribution, detailing requirements can be assessed (Wallace, 1994a; 1995).

Based on the discussion in the preceding paragraphs, it is apparent that use of a displacement-based design approach for seismic design of RC structural walls offers several advantages. First, the design of structural walls is treated in a uniform manner, without regard to arbitrary system definitions. For example, there is no need to distinguish between a bearing wall system and a dual system with respect to the proportioning and detailing of the structural wall. Secondly, the design displacement (roof displacement) can be estimated using SDOF or MDOF elastic models commonly employed in design office practice. Thirdly, specific performance objectives for a building can be assessed and implemented on a case by case basis. Finally, analytical studies (Wallace and Moehle, 1992) indicated that code provisions in effect at the time (UBC, 1991) were generally quite conservative; therefore, use of a displacement-based format would, in most cases, result in more economical wall designs.

In the following section, an overview is provided of the displacement-based design provisions for structural walls implemented in the Uniform Building Code (UBC, 1994). Based on this overview, shortcomings of the provisions are discussed and areas for future code development are outlined.

OVERVIEW OF UBC 1994 PROVISIONS

New provisions for evaluating detailing requirements for structural walls are included in UBC (1994). The provisions, primarily contained in Section 1921.6.5, were developed using a displacement-based approach by the Structural Engineers Association of California. Both a simplified procedure and a detailed design procedure, developed using a displacement-based design approach, are provided in the UBC (1994). The simplified procedure is meant to be a rapid screening technique that can be applied to walls meeting specific conditions. The detailed design approach is general in nature. These procedures are outlined in the following subsections.

SIMPLIFIED APPROACH: RAPID SCREENING

Special detailing requirements at wall boundaries are not required for geometrically symmetrical and unsymmetrical wall cross-sections with axial load less than $0.10A_gf'_c$ and $0.05A_gf'_c$, respectively, which satisfy either of the following conditions:

$$\frac{M_u}{V_u l_w} \leq 1.0 \tag{3a}$$

or

$$V_u \leq 3\,l_w t_w \sqrt{f'_c}\ \text{psi} \quad \left(0.25\,l_w t_w \sqrt{f'_c}\ \text{MPa}\right) \tag{3b}$$

where t_w is the wall thickness, A_g is the gross wall area, f'_c is the concrete strength, V_u is the design wall shear, and M_u is the design wall moment. An additional condition for Eq. (3b), shown as Eq. (4) was discussed by SEAOC (*Code*, 1993); however, this condition was not included in UBC (1994).

$$V_u \leq 3\,l_w t_w \sqrt{f'_c}\ \text{psi} \quad \left(0.25\,l_w t_w \sqrt{f'_c}\ \text{MPa}\right) \quad \text{and} \quad \frac{M_u}{V_u l_w} \leq 3.0 \tag{4}$$

For walls not meeting the conditions of Eq. (3), two alternatives are available. The first alternative applies to walls with $P_u < 0.35\ P_0$, where P_0 is the wall nominal axial load strength at zero eccentricity. These walls shall have boundary zones with special transverse reinforcement at each end of the wall a distance varying linearly from $0.25l_w$ to $0.15l_w$ for P_u varying from $0.35\ P_0$ to $0.15\ P_0$. The boundary zone shall have a minimum length of $0.15l_w$. The lateral load resistance of walls with $P_u > 0.35\ P_0$ is neglected; design is governed by requirements for deformation compatibility (UBC, 1994; Section 1631.2.4). The second alternative involves using a more detailed evaluation technique, as discussed in the following sections.

DETAILED APPROACH: DISPLACEMENT-BASED DESIGN

As an alternative to rapid screening, a more detailed procedure may be used in which calculations are required to determine the design (total) displacement in terms of the yield displacement and the inelastic displacement. The displacement at the top of the wall is obtained for the code prescribed lateral forces (reduced by R_w) using a cracked section model with either a fixed or flexible base. This result is multiplied by $3R_w/8$ to obtain the design displacement; however, care should be exercised for flexible-base models to ensure that the displacement component due to foundation rotation is not over-estimated (because base rotation is limited by the wall strength). The general process of obtaining the design displacement is depicted graphically in Fig. 4, and results in a design displacement that is equivalent to three-eighths of the maximum displacement response expected for elastic response. The wall strain distribution is determined from the wall top displacement using well established procedures (e.g., Eq. 1a). Special transverse reinforcement must be provided at the wall boundary where the compression strain exceeds 0.003. A slightly shorter minimum boundary zone length is allowed ($0.10l_w$) for the detailed evaluation compared with the rapid screening approach (minimum of $0.15l_w$).

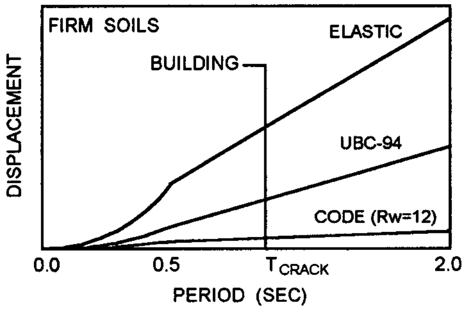

Figure 4 Code Spectral Displacement Relations: (a) Elastic, (b) Design Force Level, (c) UBC-94 Wall Design

DETAILING REQUIREMENTS

Where special transverse reinforcement is required (for walls where the extreme fiber compression strain exceeds 0.003), all vertical reinforcement in the boundary zone shall be confined by hoops or cross ties producing an area of steel not less than

$$A_{sh} = 0.09\, sh_c \frac{f_c'}{f_{yh}} \tag{5}$$

where f_{yh} is the yield strength of the hoops and cross ties, s is the spacing of the transverse reinforcement, and h_c is the cross-sectional dimension measured center-to-center of the confining reinforcement. The vertical spacing of the hoops and cross ties is limited to the smaller of 6 inches and 6 diameters of the largest vertical bar within the boundary zone. Cross ties or legs of overlapping hoops shall not be spaced further apart than 12 inches along the wall, and alternate vertical bars shall be confined by the corner of a hoop or a cross tie. The ratio of the length to width of the hoops shall not exceed 3 and all adjacent hoops shall be overlapped. The special transverse reinforcement must extend vertically a distance equal to the development length of the largest vertical bar within the boundary zone; however, this distance need not exceed l_w or $M_u/4V_u$.

ADDITIONAL REQUIREMENTS FOR FLANGED AND UNSYMMETRICAL WALLS

Special requirements were incorporated for walls with flanges or unsymmetrical cross-sections (walls with I-, T-, L-, and C-shaped cross sections). In specific, the effective flange width is limited to half the distance to an adjacent shear wall web, or 10% of the wall height (UBC, 1994, Section 1921.6.5.2). For walls that require special transverse reinforcement, the boundary zone at each end of the wall shall include the effective flange width, and shall extend at least 12 inches into the web (UBC, 1994; Section 1921.6.5.6, Paragraph 1.4).

EVALUATION OF UBC 1994 PROVISIONS

The overview of *Displacement-based Design* and the *UBC 1994 Provisions* provided in the preceding sections is used as a basis for a critical review of the UBC 1994 provisions. The objectives of the discussion that follows are to provide additional background information, as well as to inform potential users of the limitations and shortcomings of the new provisions.

DESIGN DISPLACEMENT

A critical aspect of the successful implementation of a displacement-based design approach involves obtaining a reasonable estimate of the design displacement level. The UBC (1994) procedure for estimating the design displacement at the top of the wall is equivalent to using three-eighths of the maximum displacement response expected for an elastic analysis conducted assuming stiffness quantities can be represented by cracked section values. A review of analytical studies of SDOF and MDOF systems as well as earthquake simulator studies presented earlier in this paper indicates that the UBC (1994) provisions will significantly underestimate the expected maximum displacement response, especially in the short period range. This result is depicted in Fig. 5. For "long-period" structures (with fundamental period greater than approximately 0.5 seconds for structures on rock or firm soils; Fig. 2 and 3), the UBC 1994 provisions result in design displacement values that are approximately three-eighths of those expected during strong ground motions. The discrepancies between expected and UBC (1994) displacement levels are more pronounced for "short period" structures.

This dramatic difference between the expected displacement response and that used by UBC (1994) for design will have a significant impact on the required details for structural walls. To investigate this impact, generalized results will be developed using Eq. (2). The depth of the compression zone c/l_w for a rectangular wall can be computed as a function of axial stress and extreme fiber compression strain using relationships presented by Wallace (1994a). Figure 6 plots the length of the compression zone as a function of wall length versus wall axial stress for three tension reinforcing ratios (where $\rho = A_s/l_w t_w$) using Eq. (11) from the paper by Wallace (1994a). The plot reveals that the depth of the compression zone increases with tension reinforcing ratio and axial load, and is 0.20 to $0.25 l_w$ for an axial load of $0.10 A_g f'_c$. Consider a wall where the UBC (1994) design displacement is 0.5% of the building height and the axial load is ten percent of the pure axial load capacity. According to Fig. 6, the depth of the compression zone with respect to the wall length is approximately 0.25; therefore, the compression strain at the wall boundary, computed using Eq. (2), is 0.0025. Since a strain of 0.0025 is less than the limiting strain of 0.003 specified in UBC (1994), no special transverse reinforcement is required at the wall boundary. However, since the actual drift level for the wall may be 2.67 to 10 times the UBC (1994) value, depending on the period of the structure (Fig. 2), poor wall performance is likely (since maximum concrete compression strains of 0.0067 to 0.025 could result).

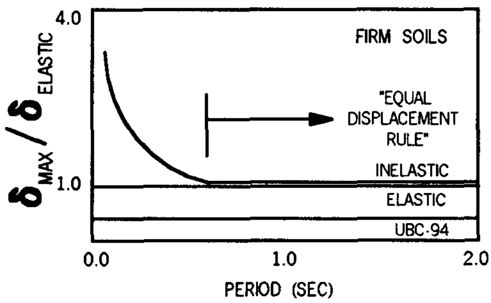

Figure 5 Displacement Response Summary

From the preceding discussion it is clear that the application of the displacement-based design provisions in UBC (1994) could lead to poor wall performance in moderate to intense ground motions due to the low estimate of the design displacement. Based on analytical and experimental studies outlined earlier in this paper, the current multiplier of $3R_w/8$ should be increased to R_w for "long-period" structures. For "short-period" structures, the maximum elastic displacement response should be amplified by a factor greater than R_w. The amplification factor can be obtained using Fig. 2, based on an estimate of the displacement ductility ratio. Displacement ductility ratios for structural wall buildings can be estimated as a function of the wall area to floor plan area provided in one direction of a building using Fig. 7 (Wallace, 1995).

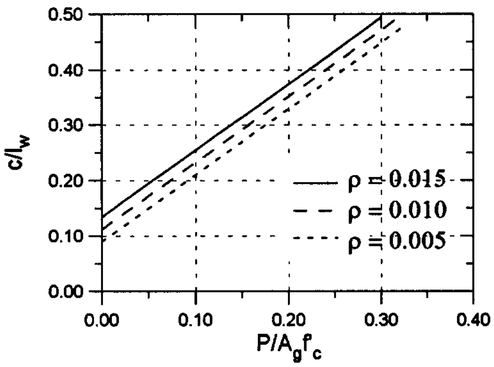

Figure 6 Wall Neutral Axis Depth

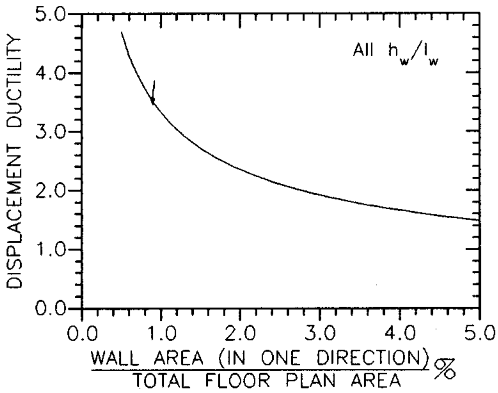

Figure 7 Displacement Ductility Ratios for Structural Wall Systems

RAPID SCREENING

The use of a versatile rapid screening approach is a good idea, since it shortens design time and may also be useful for preliminary design of new buildings and for the evaluation of existing buildings. The format used in UBC (1994) for rapid screening is convenient; however, it may be unconservative in some cases, and could be more general.

Rapid screening techniques similar in format to those used in UBC (1994) were studied by Wallace (1995). For symmetric wall cross-sections with axial load less than $0.10A_gf'_c$, the following relation for rapid screening was derived based on using f'_c = 4000 psi and code prescribed lateral forces (reduced by R_w and multiplied by 1.4 to account for a load factor):

$$\frac{v_{max}\ \text{psi}}{\sqrt{f'_c}} = \frac{120 + 35\alpha}{R_w(\alpha)^2} \tag{6}$$

where v_{max} is the maximum wall shear stress in *psi* that can be applied to the wall such that special transverse reinforcement at the wall boundary is not required, and $\alpha = M/Vl_w$, the ratio of the applied moment to the applied shear normalized by the wall length. The term R_w is needed in the expression to account for the variation of design loads for the various structural wall systems defined in UBC-1994 (e.g., bearing wall or dual systems). As mentioned previously in this paper, a displacement-based design approach allows for the uniform treatment of all structural wall systems; therefore, if code level forces are used for rapid screening, which is probably the most convenient format for design, then R_w is required to normalize the results.

For M/Vl_w equal to 3, Eq. (6) indicates a maximum shear stress of 2 and $4\sqrt{f_c'}\ psi$ for R_w equal to 12 and 6, respectively (Fig. 8). Therefore, the UBC (1994) rapid screening technique of limiting wall shear stress of $3\sqrt{f_c'}\ psi$ is not sufficient to distinguish between walls that require special transverse reinforcement and those that do not. Equation (6) suggests that wall shear stress, the ratio M/Vl_w, and the design force level must be considered to develop a comprehensive rapid screening technique. For example, coupling the limiting shear stress of $3\sqrt{f_c'}\ psi$ with a limiting M/Vl_w of 3 has been discussed (ICBO, 1993); however, this would be unconservative for $R_w = 12$ and conservative for $R_w = 6$.

Equation (6) indicates that the UBC (1994) rapid screening provision for $M/Vl_w < 1.0$ is not triggered until a wall shear stress level of $13\sqrt{f_c'}\ psi$ is reached, indicating the current provision is conservative since v_{max} of $13\sqrt{f_c'}\ psi$ is likely to exceed the shear strength of the wall and design of the wall will governed by shear and not by flexure. This suggests that a limit of $M/Vl_w < 1.5$ might be more appropriate, for which $v_{max} = 6\sqrt{f_c'}\ psi$

Based on Eq. (6) and the discussion in the previous paragraphs, it is apparent that numerous "pairs" of limiting M/Vl_w and shear stress exist for which special transverse reinforcement is not needed at the wall boundary. For example, for M/Vl_w of 2, limiting shear stress values of 4 and $8\sqrt{f_c'}\ psi$ are computed using Eq. (6) for R_w equal to 12 and 6, respectively. UBC (1994) provisions should be modified to incorporate this trade-off between R_w, v_{max}, and M/Vl_w; Equation (6) also suggests that considerable flexibility could be incorporated into this modification.

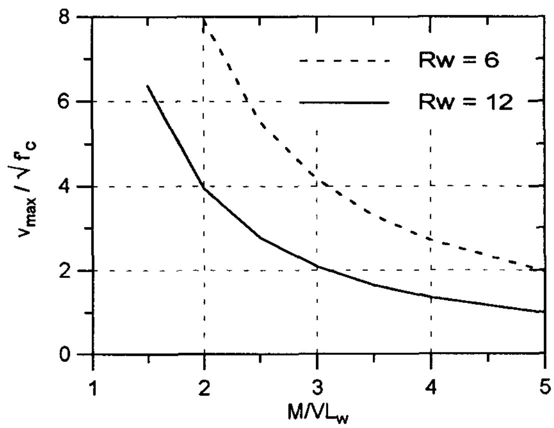

Figure 8 Wall Shear Stress Limits

Simplified rules, to determine the wall length that must be provided with special transverse reinforcement, were incorporated into UBC (1994) for walls that do not meet the rapid screening requirements. The validity of these rules can be assessed by combining Eq. (10) and (11) from the paper by Wallace (1994). Relations are plotted in Fig. 9 for three levels of axial load for a limiting extreme fiber compression strains of 0.003 even though a value of 0.004 is recommended by Wallace and Moehle (1992). A limiting strain of 0.003 is used to allow direct comparison with the UBC (1994) provisions. The value c'/l_w in Figure 9 indicates the length of the wall cross section at the wall boundary where the compressive strains exceed 0.003. The figure reveals that the UBC (1994) provisions are slightly conservative for low levels of axial load; however, they become unconservative for higher levels of wall axial load, particularly at low ratios of wall area to floor plan area. For example, for a wall area to floor plan area of 0.01 with axial loads of $0.20A_gf'_c$ and $0.35A_gf'_c$, Fig. 9 indicates a confined region of $0.20l_w$ and $0.38l_w$ should be used, respectively; In contrast, UBC (1994) allows confined regions at the wall boundary of $0.175l_w$ and $0.25l_w$, respectively. It is noted that the UBC (1994) requirement for special transverse reinforcement over $0.25l_w$ is probably a sufficient deterrent; however, designers should be aware of this discrepancy and modifications should be considered to assess the potential for unconservative designs.

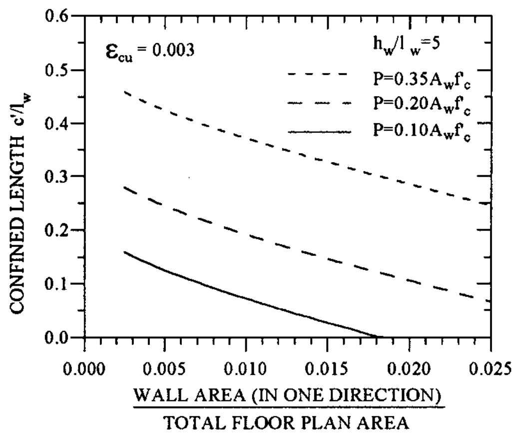

Figure 9 Wall Length Requiring Confinement

UNSYMMETRICAL AND FLANGED WALLS

UBC (1994) makes specific recommendations for the treatment of flanged walls, including the following topics: (1) the selection of effective flange widths, (2) the evaluation of detailing requirements for wall boundary zones, and (3) the description of a rapid screening technique. In the following subsections, the behavior of unsymmetrical wall cross-sections is reviewed and UBC (1994) provisions are evaluated.

Behavior of Unsymmetrical Wall Cross-Sections

The lateral load behavior of walls with unsymmetrical cross-sections differs significantly from walls with symmetrical cross-sections. For example, consider moment-curvature relations for a rectangular and T-shaped wall plotted in Fig. 10, where the T-shaped wall is formed from two of the rectangular wall cross-sections. The analyses are based on using an elastic-perfectly plastic steel stress-strain relation, an unconfined concrete stress-strain relation, and an axial load of $0.10A_gf'_c$. In addition, plane-sections are assumed to remain plane after the application of load and the effective flange width for the T-shaped wall incorporates the entire flange. Relative wall strain distributions at maximum moment (indicated by the * on Fig. 10) are plotted in Fig. 11.

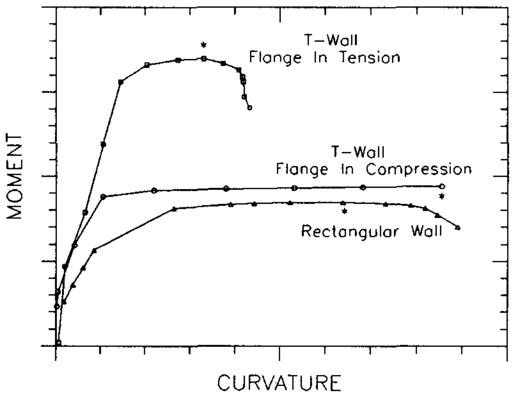

Figure 10 Wall Moment-Curvature Relations

For the given level of axial load, the rectangular wall possesses moderate deformation capacity. The behavior of the T-shaped wall depends on the direction of the applied load. When the flange of the T-shaped wall is in compression, substantial deformation capacity exists. The large deformation capacity is a result of the relatively shallow compression zone required to balance the tension force that develops at the web boundary. In contrast, when the flange of the T-shaped wall is in tension, relatively brittle behavior is noted. The brittle behavior results because the compression force developed in the wall web must balance the tension force that develops in the web boundary reinforcement, as well as the tension force that develops in the flange longitudinal reinforcement (web and boundary). Due to this large compression, special attention to detailing requirements at the boundary of the wall web is needed to provide concrete confinement and to suppress buckling of the longitudinal reinforcement.

The shear force that the wall web must resist is also affected by the shape of the wall cross-section, since much larger flexural capacity is developed when the flange is in tension. For example, a comparison of the moment-curvature relations for the rectangular and T-shaped walls in Fig. 10 indicates that shear force on the T-shaped wall is approximately twice that for the rectangular wall. Failure to consider the wall shear force likely to develop could lead to poor performance.

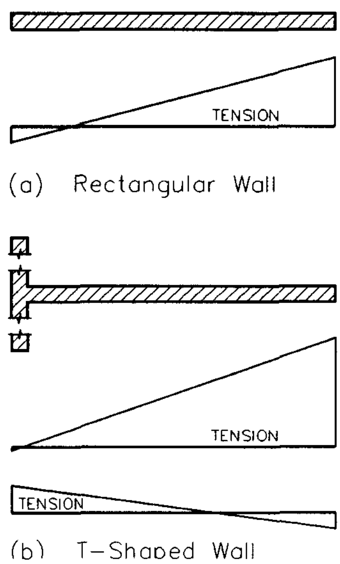

Figure 11 Wall Strain Distributions

The discussion in the preceding paragraphs is based on the assumption that the entire flange is effective in both tension and compression. An accurate assessment of the effective compression flange width is generally not required as changes in this width will have an insignificant affect on the ultimate wall strength and substantial ultimate curvature capacity exists for this case (Fig. 10). In contrast, an accurate assessment of the effective flange width when the flange is in tension is needed because only that longitudinal reinforcement within the effective flange width should be considered as tension reinforcement. The amount of tension reinforcement that is effective depends on the imposed lateral deformations on the wall, with the effective flange width increasing with increasing lateral deformation (Thomsen and Wallace, 1995), similar to reported results for T-beams with the slab in tension (Pantazopoulou and Moehle, 1987).

Experimental Studies

Experimental studies of structural walls with rectangular and T-shaped cross-sections (Thomsen and Wallace, 1994; 1995; Taylor and Wallace, 1995) have been conducted to investigate the behavior of unsymmetrical walls as well as to validate the use of a displacement-based design approach. The walls were approximately 1/4 scale and cyclic lateral loads were applied at the top of the walls (Fig. 12). A constant axial load of approximately $0.10A_gf'_c$ was maintained for the duration of the testing. Test results for three wall specimens are discussed in the following paragraphs.

Reinforcing details at the base of the walls are presented in Fig. 13 through 15. Transverse reinforcement at the wall boundaries was selected assuming a design drift level of 1.5% of the

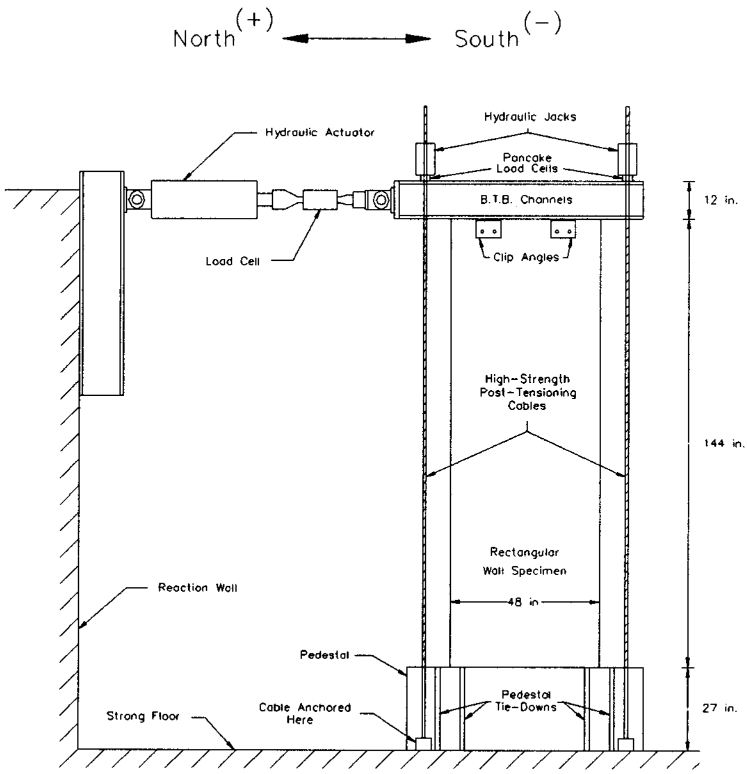

Figure 12 Test Setup

wall height using a displacement-based design procedure (Wallace, 1995). The first specimen, RW1, has a rectangular cross-section with eight #3 deformed bars at each wall boundary (Fig. 13, 14). Web reinforcement consists of deformed #2 bars and transverse boundary reinforcement consists of 3/16 in. diameter hoops and two cross-ties spaced at 3 inches ($8d_b$, where d_b is the diameter of the boundary vertical reinforcement). The second specimen, TW1, has a T-shaped cross-section. The two rectangular sections that comprise the cross-section are identical to specimen RW1; therefore, specimen TW1 represents a design where two rectangular walls that comprise the T-shaped wall are designed individually without evaluating the behavior of the combined cross-section. The third specimen, TW2, is also T-shaped; however, the detailing and shear strength requirements were evaluated based on the expected behavior of the T-shaped wall cross-section using a displacement-based design approach. The evaluation for TW2 indicated that more stringent detailing was required at the boundary of the wall web, as well, additional shear reinforcement was needed (Fig. 15). However, details were relaxed at the web-flange intersection and at the flange boundary (note that only uniaxial loads in the plane of the wall web were applied to the specimens).

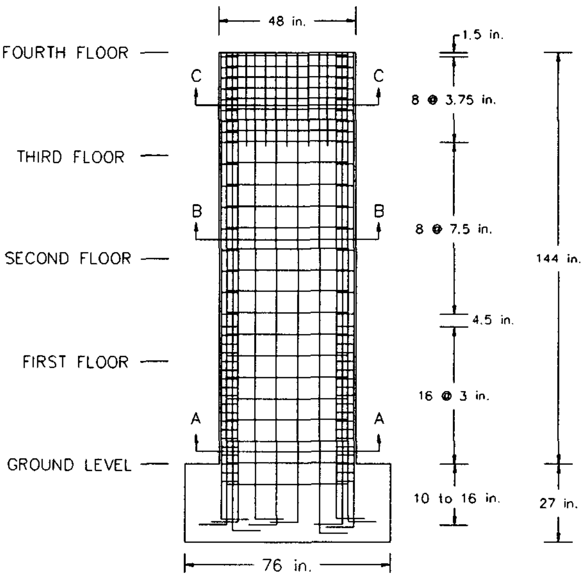

Figure 13 Geometry and Reinforcing Details - RW1

Lateral load versus lateral displacement response at the top of the wall is presented in Fig. 16 for all three specimens. Good performance was observed for specimen RW1 (Fig. 16a), as the design drift was 1.5% and the wall was subjected to two cycles of drift at 2% before failing during the first cycle to approximately 2.5% drift. The failure occurred due to buckling of the longitudinal boundary bars, which was expected based on the provided spacing of transverse reinforcement ($s = 8d_b$). Figure 16a also plots the measured lateral load versus displacement response for specimen TW1. Failure of specimen TW1 occurred abruptly due to buckling of the longitudinal and web vertical reinforcement, during the first cycle to approximately 1.25% drift. Strain gages mounted on the flange reinforcement indicated that neither the flange web or boundary steel had yielded at 1% drift. This result is consistent with the measured load-displacement relation (Fig. 16a), which shows that lateral load capacity was still increasing for negative loading at failure. Good behavior was noted when the flange was in compression (Fig. 16). The performance of specimen TW1 indicates the importance of properly considering the shape of the wall cross section on wall behavior.

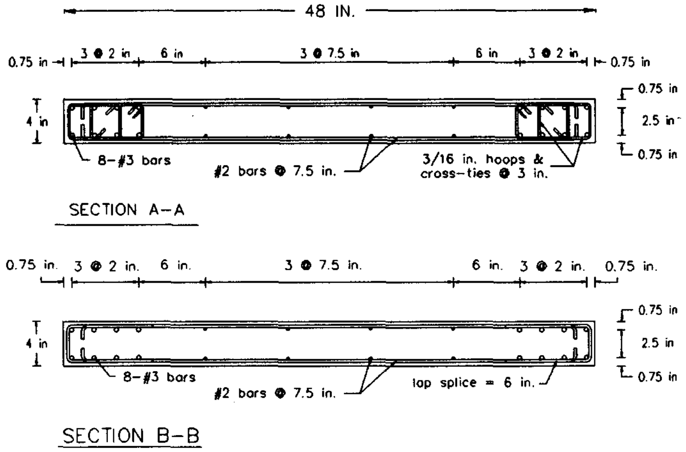

Figure 14 Geometry and Reinforcing Details - RW1

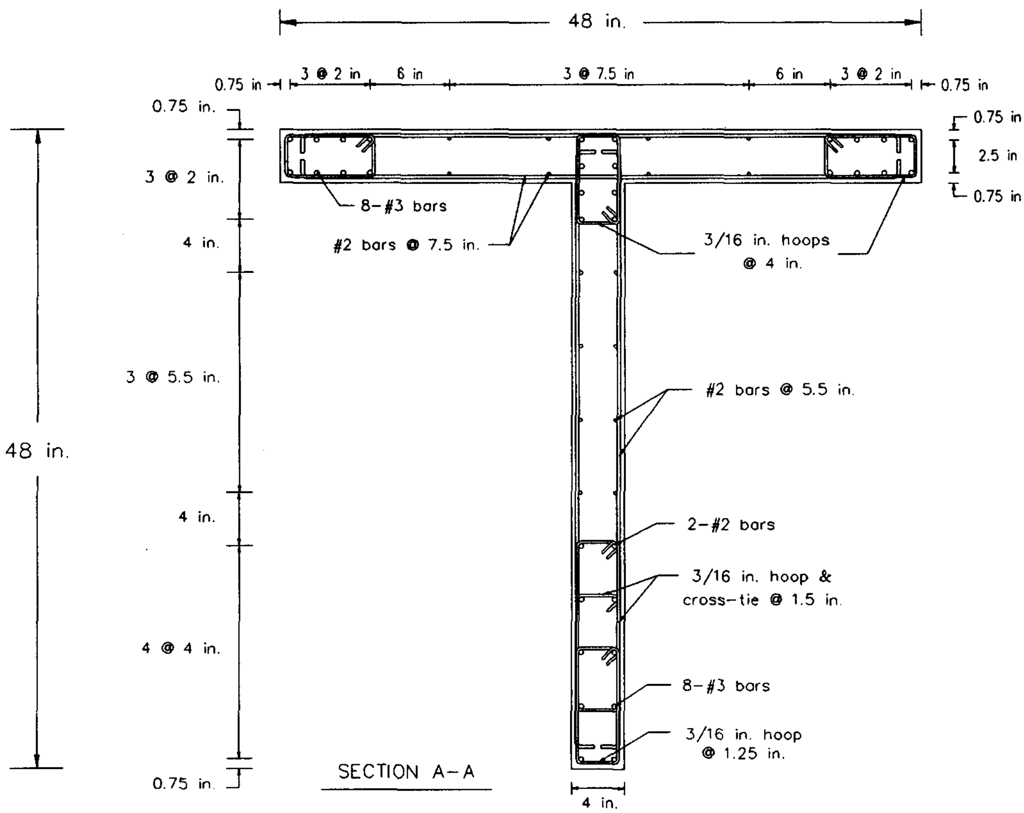

Figure 15 Geometry and Reinforcing Details - TW2

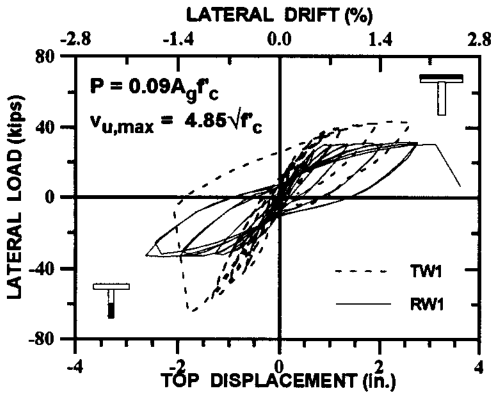

Figure 16(a) Load-Deformation Response

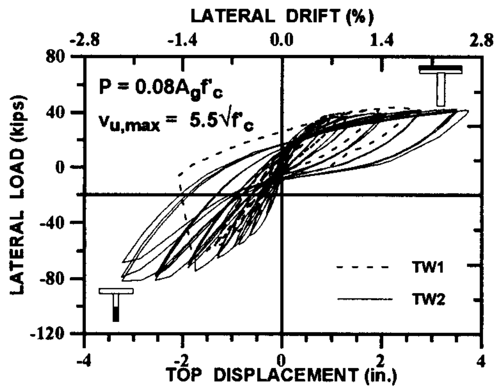

Figure 16(b) Load-Deformation Response

Figure 16b plots test results for specimens TW1 and TW2; Figure 17 plots average flange concrete strain profiles (determined using displacement gages one-half of the flange over a nine inch gage length at the base of the wall) at various drift levels for specimen TW2. Figure 16b indicates very good performance for wall TW2. The wall was subjected to a large number of displacement cycles up to 2.5% lateral drift (4 cycles at 1, 1.5, 2 and 2.5% drift) before failure occurred due to lateral web instability. This lateral web instability is not as likely for full scale walls where the cover does not make up as large a percentage of the wall thickness. A very large spacing of transverse reinforcement was used at the web-flange intersection (4 inches or $10.67d_b$, where d_b is the diameter of the boundary vertical reinforcement) and in the flange (7.5 inches or $30d_b$, where d_b is the diameter of the web distributed vertical reinforcement). Even though the flange reinforcement was subjected to significant tensile strains, the experimental results indicate that special detailing requirements for the wall flange and the web-flange intersection are not required. Maximum compression strains of 0.003 and 0.004 were measured at 1.5% and 2.5% drift.

A comparison of observed behavior for the three specimens indicates that a displacement-based approach can be used to predict both drift capacities and failure modes of the wall specimens. The studies also indicate the importance of properly evaluating unsymmetrical wall cross-sections. A critical aspect of this evaluation is estimating the effective flange width of the wall. UBC (1994) specifies that the effective flange width not exceed ten percent of the wall height, which is a relatively low value (Paulay, 1986; Wallace and Wood, 1994; Sittipunt and Wood, 1993). This provision appears to be based on the premise that it is conservative to underestimate the

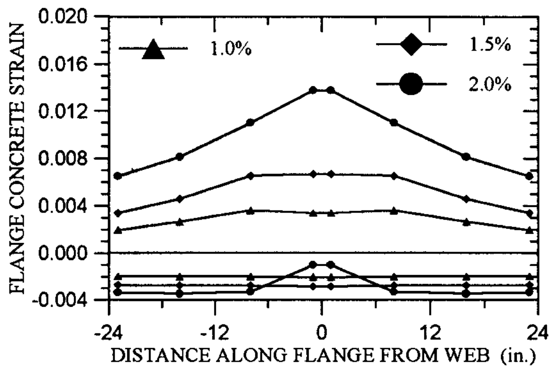

Figure 17 Flange Concrete Strain Profiles - TW2

effective flange width since it leads to a low estimate of the wall flexural strength. However, a low estimate of the effective flange width could lead to inadequate detailing of the wall web opposite the flange in tension, as well as inadequate shear reinforcement. The experimental studies outlined in the preceding paragraphs indicate that the effective flange width increases with increasing drift level, such that the entire overhanging flange of $h_w/6$ was effective at the design drift level of approximately 1.0% (once rotation of the base pedestal is accounted for, see Thomsen and Wallace, 1995); therefore, use of the UBC value of $h_w/10$ is unconservative with respect to detailing at the wall web boundary and for wall shear strength. Modification of the UBC (1994) requirements to include a more realistic estimate of the effective flange width is needed. Either a conservative value could be used (e.g., an overhanging flange with of $h_w/4$), or an effective flange width that depends on the design drift level could be implemented. Independent of what is specified in the code, good judgment should be exercised in assessing the amount of web reinforcement that should be considered as effective tension reinforcement. For example, neglecting concentrated longitudinal reinforcement lies just outside of the effective flange width specified by code requirements would not be prudent. A review of available information on effective flange widths for walls is contained in the report by Thomsen and Wallace (1995).

For unsymmetrical walls that require special transverse reinforcement at the wall boundary, UBC (1994) provisions require that the entire effective flange width be provided with special transverse reinforcement and that it extend 12 inches into the web Due to the large strain gradient that exists when the flange is in compression as well as the width of the compression zone, providing confinement reinforcement over the entire effective flange width, and especially into the wall web, is unjustified. The experimental studies of flanged walls with axial stress levels of approximately $0.10A_gf'_c$ indicate that large deformation capacities can be achieved without providing confinement reinforcement in the flange of the wall. As well, the experimental studies indicate that the strain limit of 0.003 could be increased to 0.004 with no detrimental effects since the higher compression strains occur in the concrete cover, which will not be affected by the use of confinement reinforcement.

The rapid screening technique employed in UBC (1994) for the evaluation of geometrically unsymmetrical walls does not consider the effective flange width or the amount of vertical reinforcement contained in the effective flange width. Analytical studies reported by Wallace and Moehle (1992) and by Wallace (1994a) indicate that the difference between the tension and compression reinforcement at the wall boundaries has a significant impact on wall behavior. Analytical studies by Wallace (1995) indicate that, in addition to limiting the axial load to $0.05A_gf'_c$, the difference between the tension and compression reinforcing ratios ρ - ρ' should be limited to 0.005. The reinforcing ratios should be computed as $\rho = A_s/t_wl_w$ and $\rho' = A'_s/t_wl_w$, where A'_s includes all longitudinal reinforcement in the effective flange width (the effect of concentrated flange reinforcement that falls just outside an effective flange width should also be considered), where l_w is the wall length and t_w is the wall thickness. In general, the strain distribution should be computed for each direction for an unsymmetrical section to assess detailing requirements.

WALL SHEAR STRENGTH REQUIREMENTS

UBC (1994) does not require calculations to estimate the expected wall shear force at flexural strength. This could be a significant shortcoming, especially for unsymmetrical (flanged) walls, where the actual wall shear may be several times the code specified shear force. An estimate of maximum shear force expected to develop in the wall is required to ensure adequate shear strength is provided. The maximum wall shear force can be estimated as (Paulay, 1991):

$$v_{expected} = \omega_v \frac{M_0}{M_{code}} V_{code} \tag{7}$$

where ω_v is a factor to account for the distribution of lateral force over the height of the wall at maximum shear, M_o is the flexural capacity accounting for over strength factors, M_{code} is the code required flexural strength, and V_{code} is the code specified, equivalent static shear force for a given wall.

Code required flexural strength is based on a distribution of static lateral forces that increases approximately linearly with wall height; however, due to higher mode effects, greater wall shear is likely to develop. To account for these effects, values of ω_v equal to 4/3 and 5/3 are recommended for buildings with 10 or fewer stories and buildings with more than 10 stories, respectively (Wallace, 1995). A value of ω_v of unity is appropriate where a dynamic analysis is employed. In addition, because the provided flexural reinforcement is likely to exceed that required, and to account for potential overstrength and strain hardening effects of the flexural reinforcement, an overstrength ratio (M_o/M_{code}) should be computed. The overstrength ratio can be computed for a given wall cross-section using a reinforced concrete section analysis program; however, a value of 1.4 is reasonable for symmetrically reinforced walls. A value of 1.4 is based on multipliers of 1.1 for excess reinforcement and 1.25 for overstrength and strain hardening of tension reinforcement. For unsymmetrical walls, a moment-curvature analysis should be used to determine flexural strength; however, the wall shear force can be approximated by multiplying the result obtained using Eq. (7) by the ratio of the tension reinforcement to the compression reinforcement ($\rho/\rho' > 1.0$).

The wall shear stress computed using Eq. (6) should be limited to $6\sqrt{f_c'}\ psi$ as suggested by Aktan and Bertero (1985). As well, it is noted that a shear stress of $5.5\sqrt{f_c'}\ psi$ was reached for specimen TW2, and that this level of shear stress did not significantly affect the flexural capacity or deformation capacity. For higher shear stress levels, the potential degradation of shear strength with deformation demand should be assessed (Aschheim and Moehle, 1992; Preistley et al., 1994).

SUMMARY AND CONCLUSIONS

An overview of displacement-based design and UBC (1994) provisions for reinforced concrete structural walls is provided. Based on this overview, limitations and shortcomings of the UBC (1994) provisions for seismic design of RC structural walls are presented and discussed. The following conclusions are reached:

(1) UBC (1994) provisions rely on the use of displacement values derived from code level forces (reduced by R_w) to assess detailing requirements for RC structural walls. Because different values of R_w are used depending on the definition of the structural system (e.g., bearing versus dual system), the provisions will not treat wall design in a uniform and consistent manner. Modifications to the provisions should be considered to remove this bias.

(2) By using a multiplier of $3R_w/8$, UBC (1994) provisions significantly underestimate the expected displacement response, especially for short period structures. As a result, insufficient transverse reinforcement may be provided at wall boundaries and poor wall performance may result. If displacement is to be used as a basis for design, it is imperative that a reasonable estimate of maximum displacement response be obtained. Analytical and experimental studies have been conducted that suggest a multiplier of R_w should be used for "long period" structures, and a larger multiplier is needed for "short period" structures. The multiplier for "short period" structures depends on the period and strength of the structural system. These "realistic displacement estimates" should also be used to evaluate deformation compatibility requirements.

(3) UBC (1994) provisions for rapid screening are based on limiting the ratio of M/Vl_w or the wall shear stress. These provisions may be unconservative or overly conservative because they do not properly account for the interaction of the ratio M/Vl_w, wall shear stress, and the "force reduction factor" R_w. In particular, limiting only the shear stress (Eq. 3b) is not sufficient to ensure adequate performance. The potential to draft comprehensive provisions exists using a relatively simple expression.

(4) The simplified approach provided in UBC (1994) to determine the wall length where special transverse reinforcement should be provided is unconservative for low ratios of wall area to floor plan area and for higher levels of axial load. Modifications should be considered to address this shortcoming.

(5) For unsymmetrical wall cross-sections, limiting the axial load to $0.05A_gf'_c$ is not sufficient to ensure good performance. A limit on the tension reinforcement within the effective flange width with respect to the compression reinforcement is also needed.

(6) The effective flange width specified in UBC (1994) is too low, potentially leading to poor wall performance due to crushing at the web boundary or shear distress. Experimental studies of walls with T-shaped cross-sections with a flange of approximately $h_w/6$ on each side of the web indicate that the flange width increases with imposed drift level. At a lateral drift of approximately 1.0% (given a design drift level of 1.5%), all of the tension reinforcement in the flange (web and boundary) yielded. Modifications to UBC (1994) are needed to incorporate more realistic estimates of effective flange widths. Although it is impossible to specify a single value for an effective flange width that is appropriate for

all cases, use of an overhanging flange width of $h_w/4$ is recommended. An alternative solution to this dilemma would be to use an effective flange width that varies with design displacement.

(7) UBC (1994) provisions for flanged walls require that the entire effective flange width be provided with special transverse reinforcement and that it be extend 12 inches into the web. Providing this reinforcement is costly, especially given the requirement for overlapping hoops, whereas the benefit of providing the reinforcement is questionable. Experimental studies of walls with T-shaped cross-sections indicate that detailing requirements at the web-flange intersection should be assessed by determining the wall strain distribution, and that in most cases, special detailing will not be warranted. Flange extreme fiber compression strains of 0.004 were measured with no detrimental effects on the lateral load behavior.

(8) UBC (1994) provisions do not require that the shear strength of the wall be evaluated for expected wall flexural strength, even though relatively simple procedures are available for this purpose. Implementation of these procedures should be considered, especially for unsymmetrical walls.

ACKNOWLEDGMENTS

The work presented in this paper was supported by funds from the National Science Foundation under Grant No. BCS-9112962. This financial support, as well as the support of Program Director, Dr. Shih-Chi Liu, is gratefully acknowledged. Opinions, findings, conclusions, and recommendations in this paper are those of the author, and do not necessarily represent those of the sponsor.

REFERENCES

Aschheim, M.; Moehle, J. P., "Shear Strength and Deformability of RC Bridge Columns Subjected to Inelastic Cyclic Displacements," *Report No. UCB/EERC-92/4*, Earthquake Engineering Research Center, University of California, Berkeley, (March 1992).

Bonacci, J. F., "Design Forces for Drift and Damage Control: A Second Look at the Substitute Structure Approach," *Earthquake Spectra 10(2)*: 319-332, Earthquake Engineering Research Institute, (May 1994).

Uniform Building Code, "International Conference of Building Officials," Whittier, California, (1991; 1994).

Code Change Agenda, "International Conference of Building Officials," 71ST Annual Educational and Code Development Conference, International Conference of Building Officials', Sacramento, California, (September 12-17, 1993).

Miranda, E., "Evaluation of Site-Dependent Inelastic Seismic Design Spectra," *Journal of Structural Engineering,* ASCE, Vol. 119(5): American Society of Civil Engineers, pp. 1319-1338, (May 1993a).

Miranda, E., "Evaluation of Seismic Design Criteria for Highway Bridges," Earthquake Spectra, EERI, Vol. 9(2): 233-250, (May 1993b).

Moehle, J. P., "Displacement-Based Design of RC Structures Subjected to Earthquakes," *Earthquake Spectra,* EERI, Vol. 8(3): 403-428, (August 1992).

Moehle, J. P.; Wallace, J. W., "Ductility and Detailing Requirements of Shear Wall Buildings," *Proceedings,* 5th Chilean Conference on Seismology and Earthquake Engineering, Santiago, Chile; 131-150, (August 1989).

Moehle, J. P.; Wallace, J. W.; Martinez-Cruzado, J., "Implications of Strong Motion Data for the Design of Reinforced Concrete Bearing Wall Buildings," *Report No. UCB/SEMM-90/01,* Department of Civil Engineering, University of California, Berkeley, (June 1990).

Pantazopoulou, S. J. and Moehle, J. P., "Truss Model for 3-D Behavior of R.C. Exterior Connections" *Journal of Structural Engineering,* ASCE, Vol. 116(2), pp. 298-315, (February 1990).

Paulay, T., "The design of Ductile Reinforced Concrete Structural Walls For Earthquake Resistance," *Earthquake Spectra,* EERI, Vol. 2(4): 783-823, (October 1986).

Paulay, T., "Seismic Design Strategies for Ductile Reinforced Concrete Structural Walls," *Proceedings,* International Conference on Buildings with Load Bearing Concrete Walls in Seismic Zones, French Association for Earthquake Engineering and the French Association for Construction, Paris, France, pp. 397-421, (June 1991).

Priestley, M. J. N.; Seible, F.; Xiao, Y.; Verma, R., "Steel Jacket Retrofitting of Reinforced Concrete Bridge Columns for Enhanced Shear Strength - Part I: Theoretical Considerations and Test Design," *ACI Structural Journal, 91(4)*: 394-405 (July - August 1994).

Qi, X.; Moehle, J. P., "Displacement Design Approach for Reinforced Concrete Structures Subjected to Earthquakes," *Report No. UCB/EERC-91/2,* Earthquake Engineering Research Center, University of California, Berkeley, (January 1991).

Seneviratna, G. D. P. K.; Krawinkler, H. "Strength and Displacement Demands for Seismic Design of Structural Walls," *Proceedings,* Vol. II, Fifth U.S. National Conference on Earthquake Engineering, Earthquake Engineering Research Institute, Chicago, Illinois, pp. 181-190 (July 1994).

Shibata, A.; Sozen, M. A., "Substitute-Structure Method for Seismic Design in Reinforced Concrete," *Journal of the Structural Division,* ASCE, Vol. 102(1): 1-18, (January 1976).

Shimazaki K., Sozen, M. A. (1984), "Seismic Drift of Reinforced Concrete Structures," *Research Reports,* Hazama-Gumi, Ltd., Tokyo, (1984).

Sittipunt, C.; Wood, S. L., "Finite Element Analysis of Reinforced Concrete Shear Walls," Civil Engineering Studies, *Structural Research Series No. 584,* University of Illinois, Urbana, (December 1993).

Taylor, C. P.; Wallace, J. W., "Design of Slender Reinforced Concrete Structural Walls With Openings," *Report No. CU/CEE-95/13,* Department of Civil and Environmental Engineering, Clarkson University, Potsdam, NY (December 1995).

Thomsen IV, J. H.; Wallace, J. W., "Displacement-Based Design of RC Structural Walls: Experimental Studies of Walls With Rectangular and T-Shaped Cross Sections," *Report No. CU/CEE-95/06*, Department of Civil and Environmental Engineering, Clarkson University, Potsdam, NY (June 1995).

Thomsen IV, J. H.; Wallace, J. W., "T-Shaped Shear Walls: Design Requirements and Preliminary Results of Cyclic Lateral Load Testing," *Proceedings*, Fifth U.S. National Conference on Earthquake Engineering, Chicago, Illinois, Vol. II: 891-900 (July 1994).

Wallace, J. W., "A New Methodology for Seismic Design of RC Shear Walls, "*Journal of Structural Engineering*, ASCE, Vol. 120(3): pp. 863-884, (March 1994a).

Wallace, J. W., "Displacement-Based Design of RC Structural Walls," *Proceedings*, Fifth U.S. National Conference on Earthquake Engineering, Chicago, Illinois, Vol. II: 191-200 (July 1994b).

Wallace, J. W., "Seismic Design of RC Structural Walls. Part I: New Code Format," *Journal of Structural Engineering*, ASCE, Vol. 121(1): pp. 75-87, (January 1995).

Wallace, J. W.; Moehle, J. P., "The 3 March 1985 Chile Earthquake: Structural Requirements for Bearing Wall Buildings," *Report No. UCB/EERC-89/5*, Earthquake Engineering Research Center, University of California, Berkeley, (July 1989).

Wallace, J. W.; Moehle, J. P., "Ductility and Detailing Requirements of Bearing Wall Buildings," *Journal of Structural Engineering*, ASCE, Vol. 118(6): 1625-1644, (June 1992).

Wallace, J. W.; Moehle, J. P., "An Evaluation of Ductility and Detailing Requirements of Bearing Wall Buildings Using Structural Walls," Data from the March 3, 1985, Chile Earthquake, *Earthquake Spectra*, EERI, Vol. 9(1): 137-156, (February 1993).

Wallace, J. W.; Moehle, J. P.; Martinez-Cruzado, J., "Implications for the Design of Shear Wall Buildings Using Data from Recent Earthquakes," *Proceedings*, Fourth U.S. National Conference on Earthquake Engineering, Palm Springs, California, Vol. 2: pp. 359-368, (May 1990).

Wallace, J. W.; Thomsen IV, J. H., "Seismic Design of RC Structural Walls, " *Report No. CU/CEE-93/16*, Department of Civil Engineering, Clarkson University, (August 1993).

Wallace, J. W. Thomsen IV, J. H., "Seismic Design of RC Structural Walls. Part II: Applications," *Journal of Structural Engineering*, ASCE, Vol. 121(1): pp. 88-101, (January 1995).

Wallace, J. W.; Wood. S. L., "Seismic Design of RC Shear Walls with Unsymmetrical Cross-Sections," *Proceedings*, ASCE Structures Conference, Atlanta, Georgia, April 25-27, pp. 43-48 (April 1994).

Wood, S. L.; Wight, J. K.; Moehle, J. P., "The 1985 Chile Earthquake: Observations on Earthquake Resistant Construction in Vina del Mar," *Structural Research Series No. 532*, University of Illinois, Urbana, Illinois.

Wood, S. L., "Performance of Reinforced Concrete Buildings During the 1985 Chile Earthquake: Implications for the Design of Structural Walls," *Earthquake Spectra*, EERI, Vol. 7(4): 607-638, (November 1991).

Yan, W.; Wallace, J. W., "Analytical Studies of Four Shear Wall Buildings Using Data From Recent California Earthquakes," *Report No.CU/CEE-93/15*, Department of Civil and Environmental Engineering, Clarkson University, Potsdam, NY, 141 pages, (July 1993).