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A Designer's Guide to Displacement-Based Design of RC Structural Walls

Handout

CE243A

Behavior and Design of RC Structural Elements

Fall 1998

1. INTRODUCTION

Displacement-based design of sender reinforced concrete (RC) walls provides a versatile and flexible design format for evaluating detailing requirements at wall boundaries. The basis of the approach is to use displacement as the primary design parameter to relate overall system response to wall cross section behavior. Use of a displacement-based design format does not preclude the need to evaluate wall flexural and shear strength requirements. These requirements are satisfied using well established procedures for proportioning structural elements, such as those provided in the Uniform Building Code for an equivalent static lateral force procedure or an elastic response spectrum analysis. In addition, steps must be taken to control the locations of inelastic deformations and to suppress shear failures.

2. METHODOLGY

The primary steps involved in using a displacement-based design procedure include:

1) Determine design base shear and lateral force distribution versus height for the building using established procedures such as those incorporated in the Uniform Building Code (*Uniform, 1994)* or NEHRP (1994).

2) Define the wall cross section for walls with flanges and openings.

3) Model the building and conduct an elastic analysis to determine element forces and proportion the structural elements.

4) Evaluate the likely maximum displacement response of the building, including the effects nonlinear response.

5) Calculate the curvature demand at the wall critical section due to the design earthquake (displacement estimate).

6) Evaluate detailing requirements at the critical region (usually the base) for the individual structural walls by relating the maximum displacement response of the building to the deformations that result at the wall critical section.

7) Use capacity design concepts to control the location of inelastic deformations in the wall.

8) Evaluate requirements to protect against shear failure.

To highlight the important aspects of each of these steps, consider the example building shown in Fig. 1. The building is 100 ft by 75 ft in plan and five stories tall. The story height is 12 ft and the floor dead (including tributary wall, column, and partition weight) and live loads are 150 psf and 40 psf, respectively. For purposes of this discussion, only loads in the north-south direction are to be considered; therefore, a single wall is used to provide lateral force resistance in this direction. The wall dimensions are initially selected to be 20 ft long by 2 ft wide, such that the wall aspect ratio is four and the ratio of wall area to total floor plan area is 0.0053.

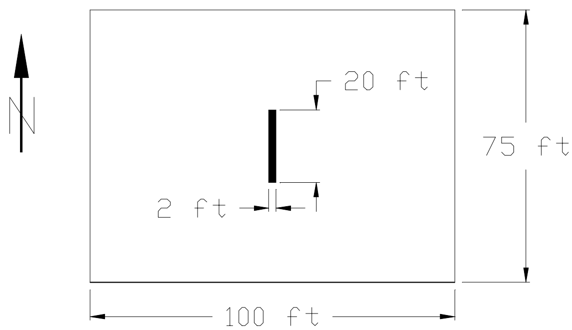

Fig. 1 Example Building Floor Plan

Step 1:

Based on UBC (*Uniform, 1994*) requirements for a building located in Zone 4 (Z=0.4, S=1. I=1, T=0.43 sec, C=2.195, R_w=8), the unfactored base shear is 0.11W, where W=5,625 kips (for a seismic dead weight of 150 psf); therefore, the unfactored base shear is 620 kips. The distribution of the lateral force over the height of the building increases linearly with values of 40, 85, 125, 165, and 205 kips at the first through fifth levels, respectively.

Step 2:

In general, the wall effective cross section must be defined for each wall in the building system. The wall cross section is clearly defined for rectangular wall cross sections; however, for walls with flanges, an effective flange width must be determined

Step 3:

UBC-94 load cases are used to assess flexural strength requirements. Assuming the wall resists the entire base shear force to simplify the analysis, the unfactored moment at the base of the wall is M = $(205^k x60')+(165^k x48') + (125^k x36')+(85^k x24')+(40^k x\ 12')$ = 27,240 ft-kips and the effective height h_{eff} of the resulting lateral load is 27,240/620 = 44 ft ($0.73h_w$). Wall reinforcing at the base should be selected to resist this moment in combination with gravity loads for appropriate load combinations. A simplified model is used to estimate required wall boundary longitudinal reinforcement (Fig. 2) which assumes a moment arm of 0.8(20 ft) = 16 ft between the tension and compression resultant in the wall and a tributary area of 1000 ft^2 at each floor level (unfactored dead load of 750 kips at the centerline of the wall). Selection of tension reinforcement at the wall boundary is controlled by the following UBC-94 load case: 0.9DL ± 1.4EQ; therefore, the tension force at the wall boundary region is T = (1.4*27,240 ft-kips - 0.9*750 kips*8 ft)/16 ft = 2,045 kips. The required area of tension steel is: A_s = 2,045 kips/(0.9*60 ksi) = 38 in^2 (24 - #11 bars or 17 - #14 bars). For this example, 14-#14 bars are provided such that an even number of bars are used. As well, since the web distributed reinforcement is neglected in the simplified analysis, the results tend to be slightly conservative. Use of a moment arm of $0.8l_w$ is also approximate, but reasonably accurate for typical walls.

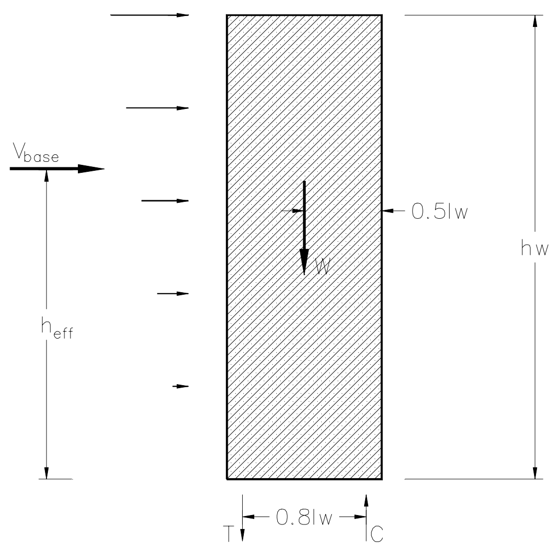

Fig. 2 Model to Estimate Wall Longitudinal Reinforcement

Because this evaluation is approximate, the design strength is verified using a moment-curvature analysis. Uniform web vertical reinforcement consists of two curtains of US #6 bars spaced at 12 inches on center (ρ=0.003). The design moment-curvature relation, for US Grade 60 steel (neglecting the effects of over-strength and strain hardening) and a concrete strength of 5 ksi, is plotted in Fig. 3. At an extreme fiber compression strain of 0.003, which is selected to be consistent with ACI 318-95 S10.2.3, the nominal moment capacity M_n is 544,700 in-kips (45,400 ft-kips) and ϕM_n is 490,230 in-kips (40,850 ft-kips), Since $\phi M_n \geq$ (M_u = 1.4*27,240 ft-kips = 38,135 ft-kips), then the wall flexural strength is adequate. It will be shown later in this report that the maximum extreme fiber compression strain is less than 0.003 for the earthquake demands placed on the wall (Fig. 11); however, the flexural strength does not vary significantly for a fairly broad range of strain for typical walls (from 0.0016 to 0.004, Fig. 3, for this

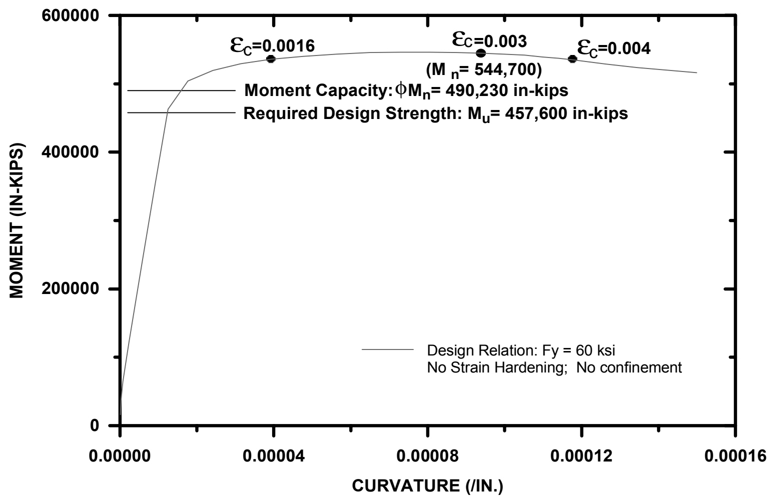

Fig. 3 Design Moment-Curvature Relation

example). At this stage of the evaluation, flexural strength may be evaluated for an extreme fiber compression strain of 0.003 subject to verification later in the evaluation process (Step 5).

Step 4:

Techniques to estimate the maximum displacement response of a given building are described in Appendix A. For this example, both a computer analysis and a simplified approach are used to assess the maximum design (roof) displacement. The simplified analysis is useful for preliminary evaluation and design. A simplified computer analysis is also presented to discuss implementation using typical structural analysis software used in consulting offices.

Building Period: The fundamental period of the building may be calculated from an eigenvalue solution using readily available computer programs provided the mathematical model accurately represents the structural properties (mass and stiffness) of the building. In addition, simplified expressions may be used to compute the fundamental period. According to UBC-94 (*Uniform,* 1994), the fundamental building period can be computed as:

$$T_g = C_t \left(h_n\right)^{3/4} \tag{1}$$

where C_t is 0.02 for reinforced concrete wall buildings and h_n is the building height in feet. A subscript of "g" is used on the period calculated using the UBC equation to denote "gross section period". To estimate displacement response, a "cracked section period" should be used (see Appendix A); therefore, the results obtained using the UBC equation should be multiplied by 1.4 based on a stiffness reduction of 50%. For this example, T_g=0.43 sec and T_{cr}=0.61 seconds.

Code Spectra: Acceleration and displacement response of building systems can be estimated using an equivalent code spectra. For Z=0.4, S=1, and I=1, the elastic acceleration and displacement spectra are plotted in Fig. 4. These figures were developed using the following equations, which are based on UBC-94 requirements (recall that the relationship between spectral acceleration and spectral displacement is: $S_d = S_a / \omega^2 = S_a T^2 / 4\pi^2$):

$$V_{base} = \frac{ZICW}{R_w} = \frac{ZI}{R_w}\left(\frac{1.25S}{T^{2/3}} \le 2.75\right)(W = Mg) \quad (2a)$$

$$V_{base} = MS_a = M\left[\frac{0.4(1.0)}{R_w}\left(\frac{1.25(1.0)}{T^{2/3}} \le 2.75\right)g\right] \quad (2b)$$

$$S_a = \left[\frac{1.1}{R_w}\right]g \qquad \text{for } T \le 0.306 \text{ sec} \quad (2c)$$

$$S_a = \left[\frac{0.4(1.0)}{R_w}\;\frac{1.25(1.0)}{T^{2/3}}\right]g = \frac{0.5g}{R_w T^{2/3}} \qquad \text{for } T > 0.306 \text{ sec} \quad (2d)$$

$$S_d = \left[\frac{1.1}{R_w}\frac{T^2}{4\pi^2}\right]\text{g} = 10.77T^2 \text{ in.} \qquad \text{for } T \le 0.455 \text{ sec} \quad (2e)$$

$$S_d = \left[\frac{0.4(1.0)}{R_w}\;\frac{1.25(1.0)T}{4\pi^2}\right]g = 4.89T \text{ in.} \qquad \text{for } T > 0.455 \text{ sec} \quad (2f)$$

Note that an exponent of 2/3 is used on the acceleration relation such that the characteristic ground period on the acceleration plot is 0.306 sec (the period at which *C > 2.75)*; this exponent is used to "artificially" require higher design forces for structures with longer periods. In contrast, this exponent is excluded from the

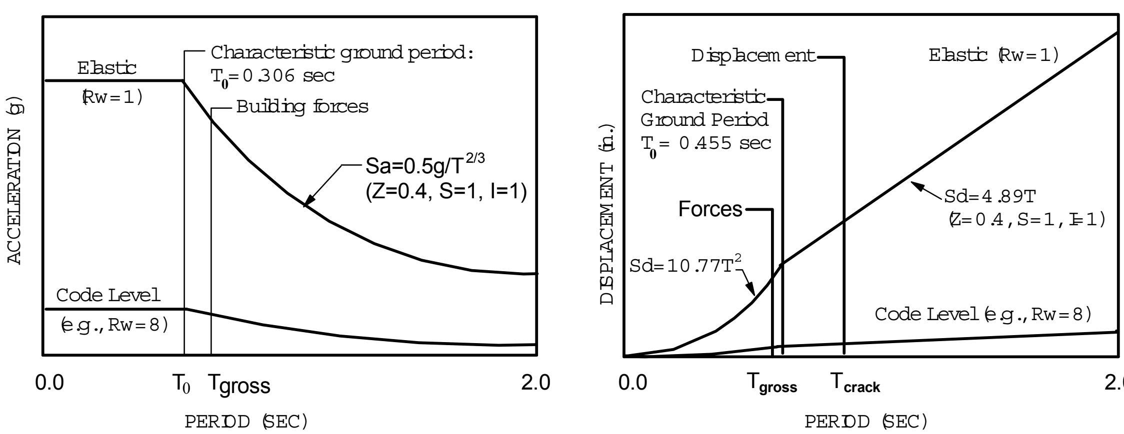

Fig. 4 Elastic Acceleration and Displacement Spectral Response

spectral displacement relations such that the characteristic period is 0.455 seconds.

For an effective period of T_{cr}=0.61 sec, the building falls in the “long period” region, that is, the fundamental period is greater than the characteristic ground period for displacement response of 0.455 sec; therefore, the maximum inelastic displacement is approximately equal to the maximum elastic displacement (see Appendix A). Therefore, elastic spectral relations can be used to estimate displacement response.

Analysis Models: In general, a two- or three-dimensional elastic response spectrum analysis could be used to assess roof displacement response. For example, consider the five story, fixed-base “stick” model of the example building shown in Fig. 5. Flexural cracking is expected in the bottom two-to-three stories; therefore, an inertia equal to 50% of the inertia based on using the gross concrete cross section is used for the bottom three stories. A value of 80% of I_g is used for stories four and five (see Appendix A). The mode shapes and the height of the resultant lateral force for the first three modes are also given in Fig. 5. The fundamental period is computed as 0.67 sec, which is very close to the “cracked” period of 0.61 sec computed using the UBC-94 equation (Eq. 1). The height of the resultant lateral force for first mode response is $0.79h_w$ or approximately equal to the height of the fifth floor level. The vector describing the first mode shape, normalized to unity at the fifth floor level, is: [0.0, 0.086, 0.31, 0.63, 1.00, 1.38]; therefore, the roof displacement is 1.38 times the displacement at the fifth level (also the height of the resultant lateral force for first mode response).

The acceleration response spectrum defined by Eq. (2a) and (2b) was used as input for the model. Displacement and base shear response for each mode and total response (all five modes) based on CQC and SRSS modal combinations are given in Table 1 (using SAP90). The CQC and SRSS combinations do not give significantly different results for this example because the modes are well separated, which is commonly the case for results of a two-dimensional analysis. The roof displacement is 3.942588 inches if all five modes are included in the response evaluation and 3.942290 if only one mode is included (CQC results). As is common for regular buildings, roof displacement is dominated by the first mode response, which contributes 99.9924% of the total response. For regular buildings, UBC-94 requires that a sufficient number of modes be included to account for 90% of the participating mass (S1629.5.1), or three modes for this example (which results in mass participation of 96.17%). Use of this provision will always capture more than 90% of the displacement response because fewer modes are required to accurately assess displacement response compared with force response (Clough and Penzien, Dynamics of Structures, 1993; Section 12.4, page 226; 1975, Section 13-4, page 202).

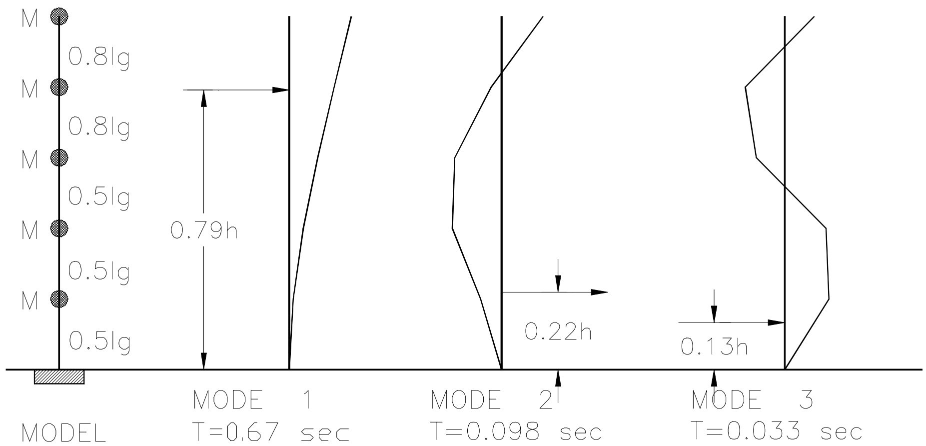

Fig. 5 Building Model and Mode Shapes

Table 1: Modal and Spectral Analysis Results: R_w=1

Mode (#)	Period (sec)	Participating Mass (%)	Roof Displacement (in.)		Base Shear (kips)	
			CQC	SRSS	CQC	SRSS
1	0.666	68.24 (68.24)	3.942290	3.942290	2523	2523
2	0.098	21.44 (89.68)	3.942587	3.942660	2852	2851
3	0.033	6.49 (96.17)	3.942588	3.942661	2882	2879
4	0.017	3.12 (99.29)	3.942588	3.942661	2889	2885
5	0.012	0.71 (100.0)	3.942588	3.942661	2889	2886

A simplified analysis, based on an equivalent single-degree-of-freedom (sdof) model, is useful to estimate displacement response for preliminary design. From the elastic displacement spectrum, the maximum displacement is S_d= 4.89(T_{cr} =0.61 sec) = 2.98 inches. Note that this response is computed for the sdof system with height equal to the height of the resultant lateral force, or 0.79h_w as shown in Fig. 5. Therefore, to determine the roof displacement, the difference between the equivalent sdof spectral displacement and the structure roof displacement for a mdof system must be taken into account. This result can be obtained using the information presented in Fig. 5 (and the first mode shape vector); therefore, the roof displacement is computed as 1.38 times the sdof displacement or 1.38(2.98") = 4.11 inches. It is noted that, for preliminary design, a detailed structural model may not yet be developed; therefore, information on fundamental (first) mode shape and the height of the resultant lateral force for first mode response are not available. To overcome this problem, parametric studies (for example, Seneviratna and Krawinkler, 1994) have been conducted to provide results that are appropriate for preliminary analysis. Values in Table 2 are recommended for use by ATC33 (75% Draft, 1995). For a five story building, a multiplier of 1.36 is recommended; therefore, a roof displacement of

1.36(2.98”) = 4.05” is calculated. It is noted that the results of the sdof model using either the multiplier determined from the stick model mode shape (1.38) or the ATC33 recommendations (1.36, Table 2) give roof displacement results that agree closely with the modal analysis of the stick model. Thus it is apparent that sdof models are extremely useful for preliminary analysis.

Table 2 Roof Displacement Multipliers for sdof Analysis Models
ATC33 75% Draft (1995) Recommendations

Number of Stories	1	2	3	4	5	10	20	>20
Multiplier	1.0	1.2	1.29	1.33	1.36	1.42	1.46	1.50

UBC-94 Displacement Estimates: UBC-94 provides procedures for computing displacement response of structural wall buildings using either an equivalent static analysis or a dynamic response analysis (response history or response spectrum). The procedure outlined for dynamic analysis is similar to that presented in this document; however, important differences exist. For UBC-94, code level displacements (for example, for R_w =8) are computed for the building model that accounts for the effects of concrete cracking. For the five story stick model, the resulting displacement response is 3.943”/8 = 0.493 inches. Detailing requirements at the wall boundaries are evaluated for a displacement response equal to the code level displacement multiplied by $3R_w/8$, or 1.48 inches. Research summarized in Appendix A (see also Wallace, 1996) indicates that the multiplier should be R_w for “long” period buildings and that a larger multiplier is needed for short period buildings. Therefore, UBC-94 provisions are potentially unsafe and use of the procedure outlined in this document is strongly recommended. Changes that address some of these shortcomings have been approved for inclusion in the UBC-97.

Influence of Foundation Flexibility: The behavior of the foundation/soil system influences the displacement response of buildings in several ways. To evaluate this impact, the foundation/soil system was incorporated into the stick model of the five story example building. A rigid footing, modeled as a beam, was assumed under the wall. Soil stiffness was modeled using vertical springs at a uniform spacing across the footing. Soil spring stiffness was arbitrarily selected to result in a fundamental period of 0.75 sec, or approximately 12% greater than the period for the fixed base model. Modal and spectral analysis results are presented in Table 3.

Table 3: Modal and Spectral Analysis Results
Fixed and Flexible Base Models

Mode (#)	Period (sec)	Participating Mass (%)	Level (#)	Displacement R_w =1 (in.)		Displacement from Base Rotation (in.)	
	Flexible	Flexible		Fixed	Flexible	R_w =1	ϕM_n (wall)
1	0.749	71.26 (71.36)	5	3.94	4.62	0.90	0.38
2	0.109	21.02 (92.38)	4	2.86	3.42	0.72	0.31
3	0.036	5.43 (97.81)	3	1.81	2.25	0.54	0.23
4	0.019	1.89 (99.70)	2	0.89	1.21	0.36	0.15
5	0.012	0.30 (100.0)	1	0.25	0.41	0.18	0.08

The rotation at the base of the wall is 0.001257 radians for R_w = 1; however, it should be noted that the base wall moment is 114,833 ft-kips which greatly exceeds the wall flexural strength of ϕM_n = 48,750 ft-kips (ϕM_n is used versus M_n such that base rotation is not over-estimated). Therefore, at a moment equal to ϕM_n, the foundation rotation is (48,750/114,833)(0.001257) = 0.000534 radians. Since the foundation/soil system is typically designed to remain elastic (with the inelastic deformations occurring within the structural wall), the foundation rotation should not exceed 0.000534 radians. Displacements at each floor level for a base rotation of 0.00534 radians are given in the last column of Table 3. The total roof displacement is 4.62 inches; therefore, the inelastic and elastic deformations must account for the difference in the total displacement and the displacement due to foundation rotation, or 4.62 - 0.38 = 4.24 inches. This value is slightly greater than the roof displacement for the fixed base model (3.94 inches). It should be noted that the damping ratio was not changed (increased) for the analysis of the flexible-base model. Damping values increase moderately for flexible-base models compared with fixed-base models due to the cyclic (hysteretic) soil behavior and radiation damping (*NEHRP, 1994).* Foundation uplift, although generally undesirable, may also act to reduce the inelastic deformations imposed on the wall. To adequately model foundation uplift and the hysteretic soil behavior requires the use of nonlinear analysis programs. Given the uncertainties associated with nonlinear analysis, the difficulties associated with evaluating and interpreting the results, as well as the time required to conduct the analysis, use of nonlinear analysis for design of new buildings is probably not warranted. Provided reasonable care is taken in the analysis and design process, use of elastic analysis with either a fixed base or flexible base (simplified model) is recommended.

Step 5:

Well established techniques can be used to assess the wall deformations required to achieve this displacement. Consider the wall system shown in Fig. 7. The displacement at the roof level can be computed as:

$$\delta_{total,\, roof} = \delta_{elastic} + \delta_{inelastic} = \delta_y + \theta_p (h_w - l_p / 2) \quad (3)$$

Where, $\delta_{elastic}$ and $\delta_{inelastic}$ are the elastic and inelastic components of the wall (roof) displacement, respectively. Since walls in reasonably configured buildings will experience yielding (Wallace and Moehle, 1992), the elastic component of displacement response may be taken as δ_y, where δ_y is the roof displacement at first yield of wall boundary reinforcement. The inelastic component of displacement response can be computed by lumping the inelastic curvature over an assumed plastic hinge length l_p at the base of the wall. *Note that this model is only valid if steps are taken to ensure that significant inelastic deformations do not occur at levels other than at the base (see step 7).* The length of the plastic hinge is selected such that the area under the actual curvature diagram is approximately equal to that for a constant curvature over the plastic hinge length. Based on this model, the displacement at the top of the wall due to inelastic deformations is equal to the plastic hinge rotation θ_p (ϕ_u x l_p) times the distance from the centroid of the plastic hinge rotation to the top of the wall.

Based on a linearly increasing distribution of lateral forces over the wall height, the displacement at the top of the wall at yield is:

$$\delta_y = \frac{11}{40} \phi_y h_w^2 \tag{4}$$

where, ϕ_y is the curvature at first yield of the wall boundary tension reinforcement For walls with axial load of approximately 0.05 to $0.15A_g f'_c$, the curvature at the base of the wall at yield can be approximated as $0.0025/l_w$ to $0.0035/l_w$ (assuming that the tension reinforcement is US Grade 60 and has yielded, and that the extreme fiber concrete compression strain is between 0.0005 and 0.0015). In general, it is wise to use a lower bound estimate of yield curvature ($0.0025/l_w$) since this will result in a conservative result by requiring greater inelastic deformations of the wall.

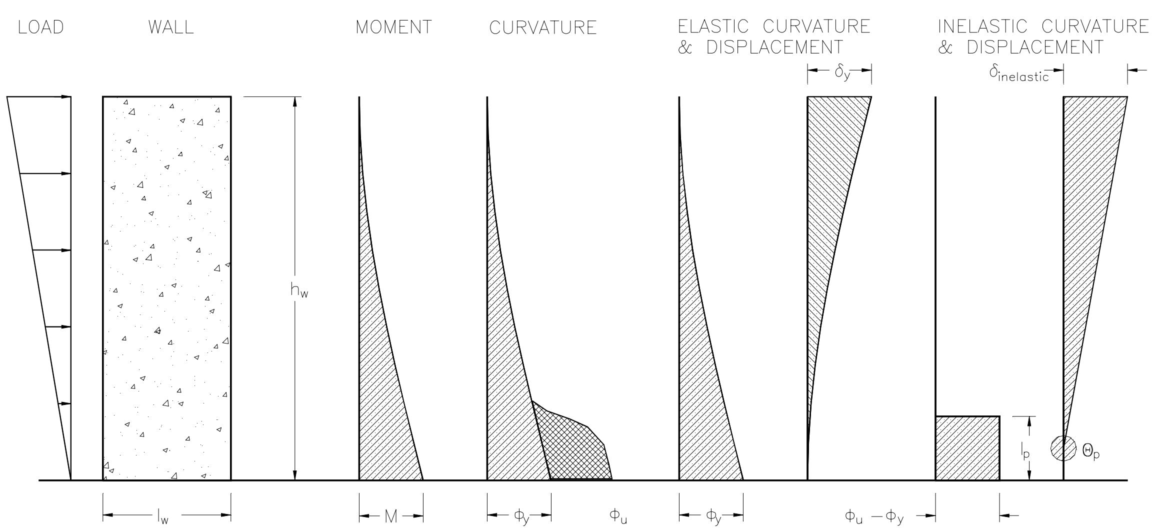

Fig. 7 Wall Modeling and Deformation Components

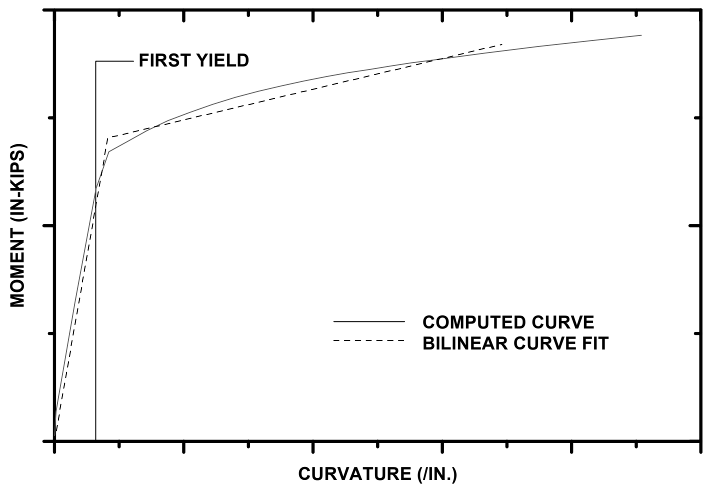

Fig. 8 Yield Point Definition for Moment-Curvature Relations

The displacement at the top of the wall due to inelastic deformations can be computed as:

$$\delta_{inelastic} = \theta_p (h_w - l_p / 2) = \left[(\phi_u - \phi_y) l_p\right](h_w - \mathrm{l_p} / 2) \quad (5)$$

where ϕ_u = the ultimate curvature developed at the base of the wall. A plastic hinge length of $0.5l_w$ is generally a good estimate for structural walls (Wallace and Moehle, 1992).

Based on these models, the ultimate curvature at the base of the wall can be computed given an estimated displacement response of the building (Step 3, Appendix B). Using an estimate of the wall yield curvature of $0.0025/l_w$, the yield displacement is estimated as 1.485 inches. Therefore, the inelastic wall deformations must account for a displacement of (5.75 - 1.485) = 4.27 inches.

$$\delta_{inelastic} = (\phi_u - \phi_y)\mathrm{l_p}(\mathrm{h_w} - l_p / 2) = (\phi_u - 0.0025 / l_w)(0.5\mathrm{l_w})(\mathrm{h_w} - 0.25 l_w) = 4.27''$$

For a wall length of 20ft (240 inches) and a wall height of 60ft (720 inches), the ultimate curvature ϕ_u at the base of the wall is 0.0000643/inch. The ratio of the ultimate curvature to the yield curvature ($0.0025/l_w$ = 0.0000104/in.), or the curvature ductility $\mu_{\phi,}$ is 6.19. The ratio of the ultimate displacement to the yield displacement, or the displacement ductility μ_δ, is 5.75/1.485 = 3.87. Note that the displacement ductility μ_δ is relatively high, in part due to the definition of the yield curvature as the curvature at first yield of the reinforcement (Fig. 8). Although this definition is appropriate for use with a displacement-based design approach, it may lead to somewhat higher displacement ductility ratios then one would expect based on a review of literature. For example, an increase of 33% in the yield displacement reduces the displacement ductility to approximately 3 for this example, which is more in line with expectations (see Wallace and Moehle, 1993).

Moehle (1992) suggested a simplified model for establishing the ultimate curvature demand at the base of the wall as:

$$\delta_{total,roof} = \phi_u l_p h_w = \phi_u (l_w / 2) h_w \qquad (6)$$

In (6), the contribution of the elastic deformations to roof displacement have been neglected and the centroid of the inelastic deformations is assumed to be at the base of the wall (versus at the centroid of the plastic hinge length). This equation is useful for preliminary design. Equation (6) can also be rearranged as:

$$\phi_u = \frac{\varepsilon_{cu}}{c} = \frac{2\delta_{total,roof}}{h_w l_w} \quad \text{or} \quad \varepsilon_{\mathrm{cu}} = 2\left(\frac{\delta_{total,roof}}{h_w}\ \frac{c}{l_w}\right) \qquad (7)$$

where $\delta_{total,\ roof}/h_w$ is the roof drift ratio and c/l_w is the ratio of the neutral axis depth to the wall length. For wall cross sections with rectangular compression zones, Fig. 9 may be used to estimate the ratio c/l_w Wallace (1996).

In Fig. 9, the wall reinforcing ratio ρ is computed as $A_s/l_w t_w$ where A_s is the area of the boundary longitudinal reinforcement. For this example, ρ=31.5 in^2/[(240”)(24”)] = 0.0055 and the axial load is 750^k/(240”)(24”)(5 ksi)=0.026; therefore, the ratio c/l_w is approximately 0.12. Substituting this into Eq. (7), with a roof drift ratio of 0.0080 (5.75”/720”), results in an extreme fiber compression strain of 0.002. The wall normal strain distribution may also be determined (given ε_{cu} and c), which is also helpful in assessing the length of the wall cross section that may need to be provided with special transverse reinforcement (see Wallace, 1995). Since the extreme fiber compression strain is relatively low for this example, no special transverse reinforcement is required for concrete confinement and requirements for transverse reinforcement are controlled by the need to restrain bucking of the boundary longitudinal reinforcement (see **Step 6)**.

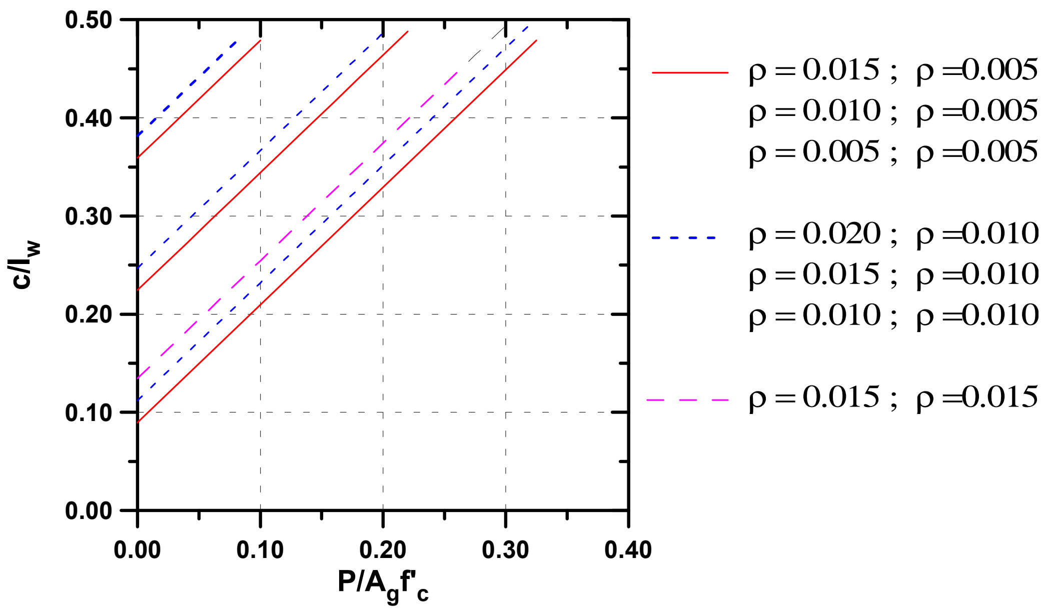

Fig. 9 Wall Neutral Axis Depth

The preliminary analysis to determine the maximum wall compressive strain expected for the design earthquake assists in assessing detailing requirements. This topic is addressed in **Step 6**.

Step 6:

Now that the ultimate curvature and maximum compression strain that the wall will be subjected to have been estimated, required transverse reinforcement at the wall boundaries (at the wall base over the plastic hinge length) to provide concrete confinement and to suppress buckling of the boundary longitudinal reinforcement can be evaluated. These items are discussed in the following paragraphs.

Requirements for concrete confinement can be assessed by using a moment-curvature analysis (Thomsen and Wallace, 1995). Realistic models for both reinforcement and concrete should be used in this analysis. For reinforcement, the effects of steel over-strength and strain hardening should be included when assessing detailing requirements since both of these parameters tend to be detrimental to wall deformation capacity. Numerous models for concrete confinement exist in the literature (for a partial listing see the manual for the computer program BIAX; Wallace and Ibrahim, 1996); however, one of the more comprehensive models developed is the one by Saatcioglu and Razvi (1992). The model accounts for the size, spacing, and distribution of transverse reinforcement as well as the distribution of longitudinal reinforcement. Variations in confining pressures that may result for rectangular cross sections are also accounted for by the Saatcioglu and Razvi model.

Several programs are available to compute moment-curvature response of RC sections (NISEE, 1996). The program BIAX (Wallace and Ibrahim, 1996) was used in the following analyses since it was readily available, incorporates the confined concrete model developed by Saatcioglu and Razvi (1992), and operates using a graphical user interface within a windows environment.

Models for confined concrete depend on the amount and distribution of transverse reinforcement used at the wall boundary - which is what needs to be determined; therefore, an iterative approach must be used. Although this would appear cumbersome, simple techniques may be used to provide a starting point (such as estimating the extreme fiber compression strain as outlined in the preceding section, **Step 5**). Given an estimate of the maximum compression strain and the depth of the compression zone, three general levels of detailing are appropriate for structural walls:

(1) high level for $\varepsilon_{cu} > 0.004$, where the maximum spacing should not exceed 6" or $6d_b$ and the requirements of UBC-94 Section 1921.6.5.6 (subparagraphs 2.1 and 2.5) should be satisfied. These requirements state the following:

2.1 The ratio of the length to the width of the hoops should not exceed three. All adjacent hoops shall be overlapping.

2.2 All vertical reinforcement within the boundary zone shall be confined by hoops or cross ties producing an area of steel not less than: $A_{sh} = 0.09 s h_c f'_c / f_{yh}$.

2.3 Hoops and cross ties shall have a vertical spacing not greater than the smaller of 6 inches or 6 diameters of the ~~largest~~ smallest vertical bar within the boundary zone.

2.4 Cross ties or legs of overlapping hoops shall not be spaced further apart than 12 inches along the wall.

2.5 Alternate vertical bars shall be confined by the corner of a hoop or a cross tie.

Maximum compression strain levels greater than 0.01 are not recommended, although UBC-94 allows strains up to 0.015.

(2) moderate level for $0.002 < \varepsilon_{cu} \leq 0.004$, where the maximum spacing should not exceed 8" or $8d_b$ and the requirements of ACI 318-95 S7.10.5. Moderate level details are intended to suppress buckling of longitudinal bars as well as to provide moderate toughness to the wall.

(3) low level for $\varepsilon_{cu} \leq 0.002$, where the maximum spacing should not exceed 12" or $12d_b$ and the requirements of ACI 318-95 S7.10.5 should be satisfied.

Assuming that moderate level details are appropriate for this wall (Fig. 10), the moment curvature relation was computed and is plotted in Fig. 11. For comparison, two relations are plotted in Fig. 11. One curve (probable) includes

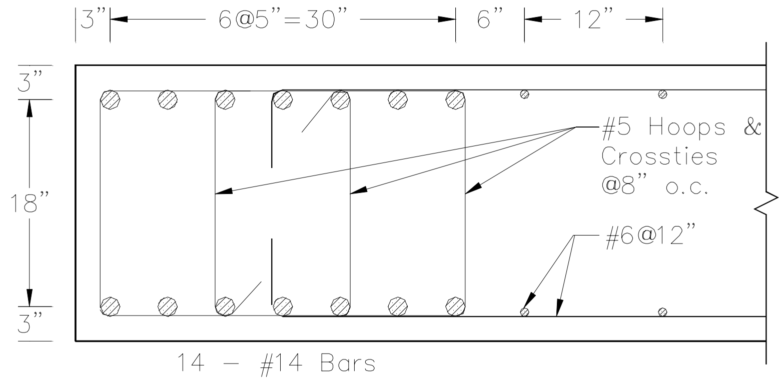

Fig. 10 Wall Boundary Reinforcement at Base of Wall

the influence of steel over-strength, steel strain hardening, and concrete confinement using the Saatcioglu and Razvi model; the second curve (design) does not include these factors.

The results plotted in Fig. 11 indicate that the wall has sufficient curvature capacity to meet the curvature demand from the design earthquake. In fact, the plots indicate that confinement is not required to meet the deformation demands imposed by the earthquake. The maximum concrete compressive strain at the extreme fiber is 0.0026 for the relation based on probable wall moment-curvature response; therefore, for this relatively low compression strain, transverse reinforcement at the wall boundary is controlled by buckling versus concrete confinement. It is also noted that this extreme fiber compression strain is close to that estimated using the approximate relations in **Step 5**;therefore, the

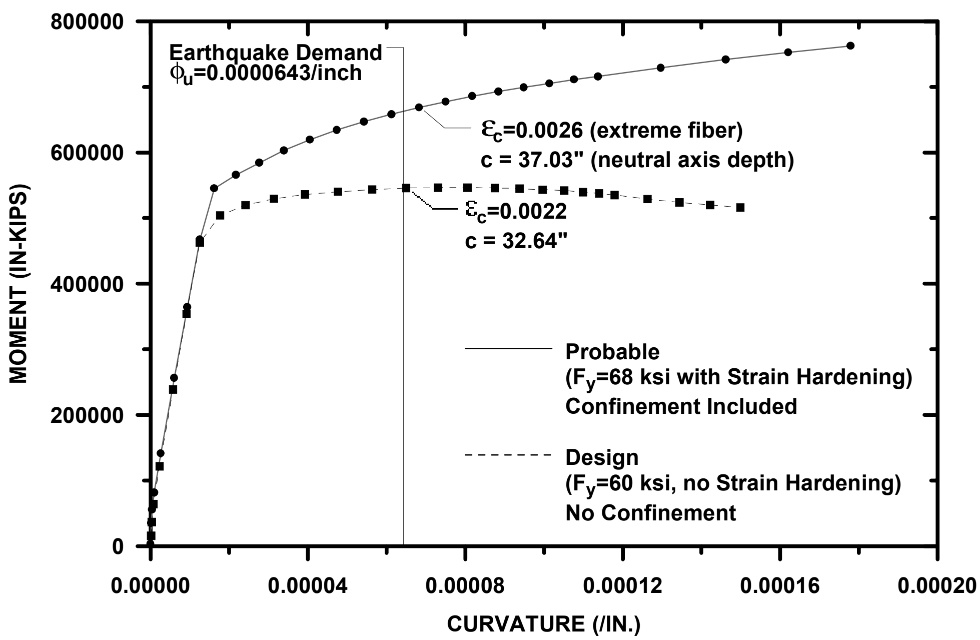

Fig. 11 Moment-Curvature Relations

preliminary selection of transverse reinforcement is adequate.

Special transverse reinforcement should be provided at the base of the wall over a height equal to the “plastic hinge length” (the region at the base of the wall where inelastic deformations are concentrated). This height may be taken as the greater of l_w or $h_w/6$. Above the plastic hinge region, hoops and ties meeting ACI 318-95 S7.10.5 requirements should be provided (vertical spacing shall not exceed 16 longitudinal bar diameters, 48 tie bar diameters, or the least dimension of the compression member).

In the plastic hinge region, horizontal web reinforcement should be hooked into the boundary region with a standard 90 or 180 degree hook. Outside of the plastic hinge region, U-shaped bars may be lapped with horizontal web reinforcement. These details have been shown to adequate for web shear stress levels less than $6\sqrt{f_c^{'}\ psi}$ (Thomsen and Wallace, 1995).

STEP 7:

Use of a displacement-based design procedure is valid only if the inelastic deformations occur primarily at the base of the wall in the well detailed region; otherwise, the basis for establishing curvature and strain demands at the base of the wall outlined in Step 5 are not valid. As well, due to uncertainties in load distribution over the height of the building (higher mode effects), material properties, and quality of construction, some measure of “conservatism” should be employed to ensure that significant inelastic actions do not occur at other levels. For uniform cantilever walls, achieving this goal is relatively easy. For walls with openings or setbacks over the height of the building, additional computation effort may be necessary to avoid undesirable behavior.

For walls with a uniform cross section over the building height, two general approaches may be used to ensure the majority of inelastic action occurs at the base of the wall. Both approaches involve using a capacity design approach. In the first approach (Paulay, 1986; 1991), the wall moments above an assumed plastic hinge region at the base of the wall (the greater of l_w or $h_w/6$) are amplified using a linear envelop for required strength. This approach is relatively straight forward and does not require substantial effort. An alternative approach (ACI Committee 368, 1996) is based on amplifying the code prescribed moments (factored) for levels above the plastic hinge region at the base of the wall as:

$$M_u(amplified) = \alpha_f \frac{M_{np,\,base}}{M_{u,\,base}} M_u \le M_{np,\,base} \tag{9}$$

where $M_{np,base}$ is the nominal (probable) moment capacity at the base of the wall, including the effects of over-strength and strain hardening of the reinforcement, at the curvature demand for the design earthquake (Fig. 11), $M_{u,base}$ is the factored moment at the base of the wall for the prescribed code forces, M_u is the factored code moment over the height of the wall for the code prescribed lateral forces, and α_f is a scale factor to provide flexural over-strength at levels above the base so as to avoid inelastic actions at these levels (a value of 1.4 is suggested). For the moment-curvature analysis depicted in Fig. 11, $M_{np,base}$ = 650,000 in-kips (54,167 ft-kips) and $M_{u,base}$ is 457,600 in-kips (38,135 ft-kips) as computed in **Step 3**. The resulting wall moment distribution is given in Fig. 12. The moment is constant at 54,167'k over the bottom 12.6 ft of the wall and then decreases at upper levels. A capacity design approach using factored moments derived from analysis is advantageous because all the information required is readily available to the designer. However, it is clear that the two approaches discussed here do not give significant differences since the design level moment distribution above the plastic hinge region is nearly linear. The broken line shown on the design level moment distribution in Fig. 12 represents the results obtained using the approach recommended by Paulay (1986; 1991), where the moment is constant over the bottom 10 ft of the wall.

STEP 8:

Although not required by UBC-94, the wall shear forces expected to develop at wall flexural capacity should be estimated to ensure adequate wall shear strength is provided. This is an especially important step for unsymmetrical (flanged) walls, where the actual wall shear forces may be several times the code specified shear forces. An estimate of maximum shear forces expected to develop in the wall is required to ensure adequate shear strength is provided

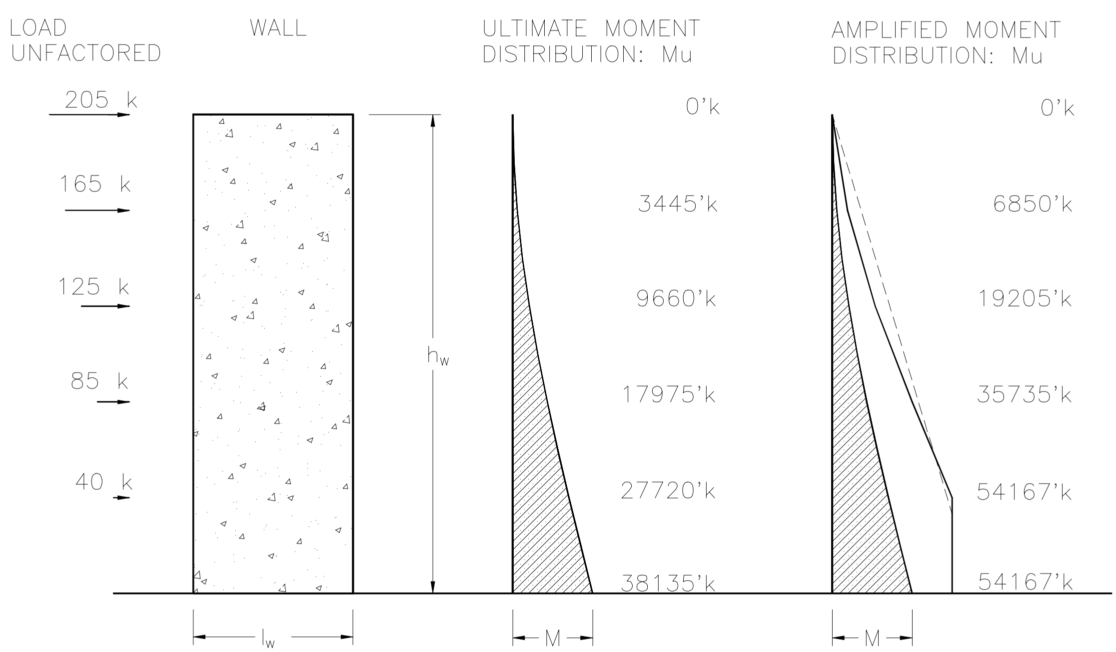

Fig. 12 Moment Distribution for Capacity Design

over the height of the wall. The amplified wall shear force, V_u (amplified), can be estimated as (Paulay, 1986):

$$V_u \text{ (amplified)} = \alpha_v \; \omega_v \; \frac{M_{np,base}}{M_{u,base}} \; V_u \tag{11}$$

where ω_v is a factor to account for the distribution of lateral force over the height of the wall at maximum shear, $M_{np,base}$ is the nominal (probable) moment capacity at the base of the wall, including the effects of over-strength and strain hardening of the reinforcement, at the curvature demand for the design earthquake (Fig. 11), α_v is a scale factor to provide shear over-strength to promote flexural “failure” prior to shear “failure”, $M_{u,base}$ is the factored moment at the base of the wall for the prescribed code forces, and V_u is factored shear force over the height of the wall for the code prescribed forces. The factored shear forces for the code prescribed lateral forces, V_u, may be determined using an equivalent static or modal analysis. The value selected for α_v depends on whether flexural yielding is expected prior to reaching the shear strength of the wall, which can be evaluated by comparing moment-curvature response at the expected earthquake curvature demand with the ratio of wall shear force at this curvature to the shear capacity. For this example, flexural yielding is expected; therefore, a value of α_v of 1.0 is suggested since flexural over-strength has been accounted for in Eq. (11). For walls controlled by shear, α_v of 1.3 is suggested.

If an equivalent static analysis is used, the distribution of code prescribed lateral forces increases roughly linearly with wall height; however, due to higher mode effects, greater wall shear is likely to develop. To account for this phenomenon, 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 and sufficient modes are considered to capture higher mode effects (for example, sufficient modes are included to capture 90% of the participating mass). The ratio ($M_{np,base}/M_{u,base}$) in Eq. (11) accounts for the influence of excess longitudinal reinforcement, as well as the impact of reinforcement over-strength and strain hardening on wall shear. The over-strength ratio can be computed for a given wall cross-section using a reinforced concrete section analysis program; however, for symmetric walls that are not significantly over-designed, a value of 1.5 is a reasonable estimate that could be used for preliminary design. A value of 1.5 is based on multipliers of 1.15 for excess reinforcement and 1.30 for over-strength and strain hardening of tension reinforcement. For unsymmetrical walls, a moment-curvature analysis should be used to determine flexural strength. The wall shear force computed using Eq. (11) should be limited to $6A_{cv}\sqrt{f_c' \;\, psi}$ as suggested by Wallace (1995) based on the evaluation of wall tests as reported by Aktan and Bertero (1985) and Wood (1990). It is noted that this is significantly less than that currently allowed in UBC-94 (or ACI 318-95), where shear forces of $10A_{cv}\sqrt{f_c' \;\, psi}$ and $8A_{cv}\sqrt{f_c' \;\, psi}$ are allowed on a single pier and a group of piers resisting a common lateral force, respectively. However, experimental results do not support use of these values. As an upper limit, values of $8A_{cv}\sqrt{f_c^{'}\ psi}$ and $6A_{cv}\sqrt{f_c^{'}\ psi}$ should be used in place of $10A_{cv}\sqrt{f_c^{'}\ psi}$ and $8A_{cv}\sqrt{f_c^{'}\ psi}$, respectively; however, use of $6A_{cv}\sqrt{f_c^{'}\ psi}$ is recommended as good design practice. This issue is currently being discussed within ACI Committee 318-H. It is likely that use of $6A_{cv}\sqrt{f_c^{'}\ psi}$ will be viewed as too costly (putting concrete at an economic disadvantage); therefore, it is more likely that $8A_{cv}\sqrt{f_c^{'}\ psi}$ and $6A_{cv}\sqrt{f_c^{'}\ psi}$ will emerge as a compromise.

For this example, the ratio of $M_{np,base}/M_{u,base}$ is 54,167/38,135 = 1.42, V_u(base) = 1.4(620) = 868 kips, and ω_v = 4/3. For α_v of 1.0, the amplified shear force and shear stress at the base of the wall are 1640 kips and 285 psi $\left(4.03\sqrt{f_c^{'}\ psi}\right)$, respectively; therefore, the wall shear stress level is acceptable.

Shear reinforcement should be selected using ACI 318-95 (“Building,” 1995) requirements. According to ACI 318-95 Eq. (21-6), the nominal shear strength of walls with aspect ratio $h_w/l_w \geq 2.0$ is:

$$V_n = A_{cv}\left(2\sqrt{f_c^{'}} + \rho_n f_y\right) \tag{12}$$

where A_{cv} is the area bounded by the web and the wall length ($l_w t_w$) and ρ_n is the shear reinforcing ratio. In general, equal amounts of horizontal and vertical web reinforcing should be used; therefore, ρ_n may be calculated as either the vertical or horizontal web reinforcing ratio. Based on the flexural design, two curtains of #6 US Grade 60 bars spaced at 12 inches on center were selected to give a web reinforcing ratio of 0.00306. Shear design requirements are based on $\phi V_n \geq V_u$(amplified), or (0.85)(240”)(24”)[2(5000 psi)$^{1/2}$ + ρ_n(60,000 psi)] ≥ 1,640,000 pounds. The required spacing for two curtains of #6 US Grade 60 bars is 11.35 inches ($\rho_n \geq 0.00323$); therefore, a spacing of 10 inches is used (ρ_n = 0.00367) for which ϕV_n = 1770 kips.

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APPENDIX B: DISPLACEMENT RESPONSE OF BUILDING SYSTEMS

In a displacement-based design format, the displacement response of the building system is related to the resulting wall curvature or normal strains; therefore, a critical aspect of using a displacement-based design process involves estimating the maximum displacement response of a building system. 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, Moehle, and Martinez-Cruzado, 1990; 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) sdof and mdof building models on earthquake simulators, and (3) instrumented buildings subjected to earthquake ground motions. 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. B.1, 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. B.1). It is noted that the relations plotted in Fig. B.1 were computed using a elastic-perfectly plastic force-deformation model; therefore, incorporation of moderate strain hardening would result in slightly lower deformation demands, particularly in the short period range.

Similar trends to those reported by Miranda (1993) are reported for studies sdof and mdof systems tested on earthquake simulators (Shimazaki and Sozen, 1984; Bonacci, 1994; Fig. B.2). For the earthquake simulator tests, if the initial (fundamental) period of the building model including the effect of cracking ($T_{cracked}$) exceeds the characteristic ground period T_0, 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_0, the maximum inelastic displacement response exceeds that for an elastic system (Fig. B.2). The characteristic ground period can be estimated as the period where the constant acceleration region and constant

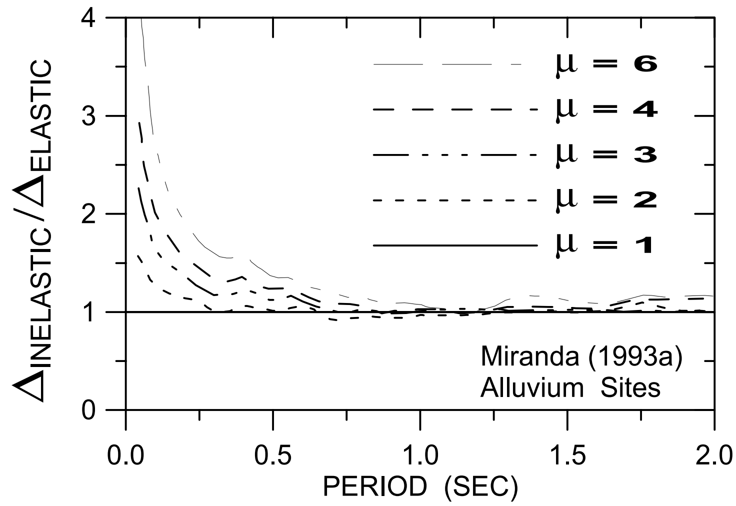

Fig. B.1 Mean Ratio of Inelastic Displacement Response to Elastic Displacement Response of SDOF Systems

velocity region coincide on a 4-way log plot of spectral ordinates, and is approximately 0.5 sec for the results reported in Figure B.2.

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

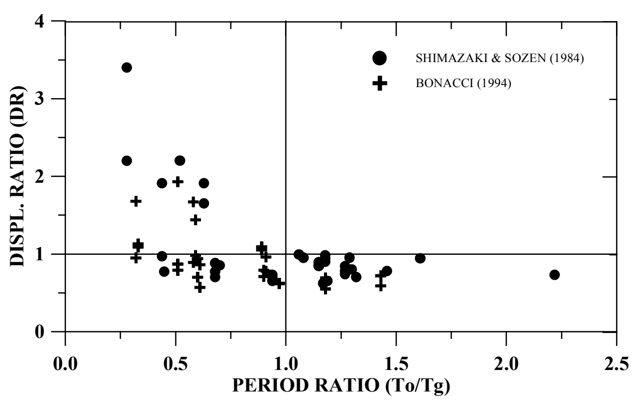

Fig. B.2 Displacement Response Ratios for Earthquake Simulation Studies

Miranda (1993a) based on estimates of the initial period of the structure (accounting for cracking) and the anticipated level of inelastic response (displacement ductility).

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) should be used (Fig. B.3). For evaluating detailing requirements, a response spectrum analysis based on using an effective stiffness and an unreduced spectrum (R_w =1) should be used (Fig. B.3).

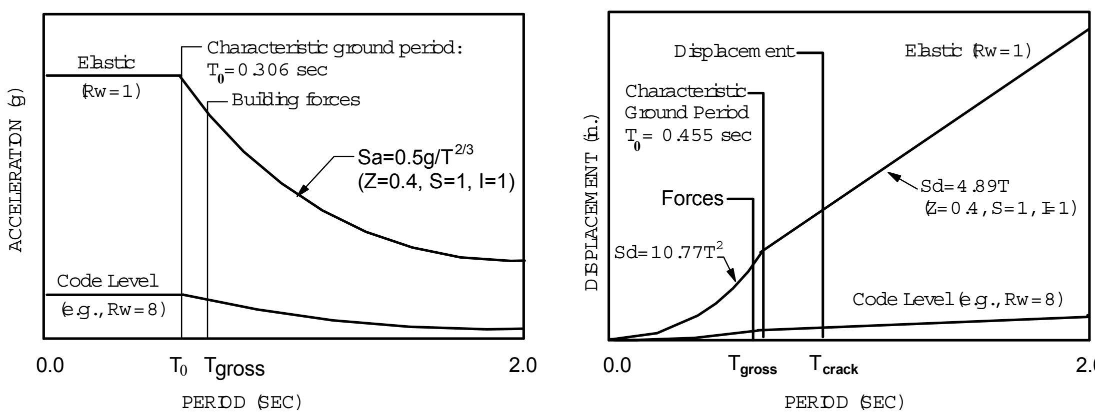

Fig. B.3 Design Response Spectra (see Fig. 4)

Modifications in the short period range should be applied based on the information presented in the preceding paragraph (Fig. B.1 and B.2). In addition, ATC 33 (Guidelines, 1995) outlines a procedure for estimating the displacement response of “short period” buildings. This procedure is outlined in the following paragraph (note that the ATC 33 document is not yet complete and thus cannot be referenced at this time). According to ATC 33 75% draft guidelines the displacement computed for elastic response should be multiplied by :

$$C_1 = 1.0 \quad \text{for } T_{cracked} \geq T_0$$

$$C_1 = \left[(R-1)T_0 / T + 1\right] / R \leq 2 \quad \text{for } T_{cracked} < T_0$$

where S_a is the spectral acceleration, $R = \dfrac{S_a / g}{(V_y / W) C_0}$, W is the seismic dead weight of the building, V_y is the yield level for a bilinear relation that approximates the base shear versus roof displacement response (similar to what is shown in Fig. 6 for moment-curvature response), and C_0 is a factor that accounts for the difference between the sdof spectral displacement and the building roof displacement. In decreasing order of accuracy, it may be obtained as

1. the modal participation factor using a shape vector corresponding to the deflected shape of the structure at the target (expected maximum) displacement level,
2. the modal participation factor for the first elastic mode of the structure,
3. the appropriate value from the following table.

Number of Stories:	1	2	3	4	5	10	20	>20
Values of C_0	1.0	1.2	1.29	1.33	1.36	1.42	1.46	1.50

Finally, recommendations are needed to address what are appropriate levels of cracking to consider for elastic response. 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. ATC 33 75% draft guidelines provide recommendations for other structural elements in Table 6.3, as summarized in Table C.1.

Table C.1 Effective Stiffness Values for Structural Analysis

Component	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Beams: non-prestressed	$0.5E_cI_g$	$0.4E_cA_w$	E_cA_g
Beams: prestressed	$1.0E_cI_g$	$0.4E_cA_w$	E_cA_g
Columns in compression	$0.7E_cI_g$	$0.4E_cA_w$	E_cA_g
Columns in tension	$0.5E_cI_g$	$0.4E_cA_w$	E_cA_g
Walls: uncracked	$0.8E_cI_g$	$0.4E_cA_w$	E_cA_g
Walls: cracked	$0.5E_cI_g$	$0.4E_cA_w$	E_cA_g
Flat Slabs: non-prestressed	$0.35E_cI_gK_{fp}$	$0.4E_cA_g$	E_cA_g
Flat Slabs: prestressed	$0.5E_cI_gK_{fp}$	$0.4E_cA_g$	E_cA_g

I_g for T-beams may be taken as twice the value of I_g of the web alone, or may be based on the effective section using ACI 318 Section 8.10.

K_{fp} = 1.0 at interior supports, 0.8 at edge and exterior supports, and 0.6 at corner supports.

For shear rigidity, the quantity $0.4E_c$ has been used to represent the shear modulus G.

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 (STEP 4).

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