RC STRUCTURAL WALLS DESIGNED ACCORDING TO UBC AND DISPLACEMENT-BASED METHODS

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ABSTRACT: The seismic provisions of the 1997 *Uniform Building Code* (UBC) are examined from the perspective of achieving performance-based earthquake engineering of structural wall buildings. It is shown that although strain limits are present in the 1997 UBC, the drift ratio limits generally govern design. The conflict between assumed force reduction factors and actual ductility demand at the design limit state controlled by drift is also explored. Through the use of design examples and dynamic inelastic time history analysis it is shown that attempts to achieve performance-based engineering with a force-based approach such as that described in the 1997 UBC will inevitably not be possible. As an alternative, it is shown that a simple and more rational direct displacement-based approach would better achieve the objectives of performance-based earthquake engineering.

INTRODUCTION

Since approximately 1990, a significant movement in the earthquake engineering community has been directed towards performance-based seismic design or limit-states design. The philosophy has the simple aim of specifying structural performance for one or more earthquake intensity levels.

The simple objective of specifying structural performance has, however, been a significant source of discussion. As defined by the SEAOC (1995) *Vision 2000*, performance-based engineering contains several facets, which include definition of performance levels, conceptual design, structural design, design verification, design review, and quality control. The areas that generate the majority of discussion are (1) definitions of performance levels and performance indicators and (2) structural design methods for achieving performance-based design.

For the last several decades, seismic design has been performed with what is often termed ''force-based design,'' which has worked well and generally met the objective of achieving a safe design. However, with the increased interest in performance-based earthquake engineering, the future of design methods such as force-based design must be questioned.

Given that the primary objective of performance-based earthquake engineering is achieving predefined levels of damage for specified earthquake intensity levels, any design procedure used in the process must be capable of reliably controlling damage. Due to the familiarity and relative simplicity of force-based design, there have been attempts to achieve aspects of performance-based earthquake engineering with force-based design. One such example can be found in the 1997 Uniform Building Code (UBC) (ICBO 1997).

Due to the fundamental characteristics of force-based design such as (1) use of simplified relations between elastic and inelastic displacements, (2) application of behavior modification or force-reduction factors, and (3) assumption of strength independent of stiffness, it will be shown that efforts at achieving performance-based design with force-based methods are essentially futile, unless combined with more rigorous analysis methods. It is the objective of this paper to investigate the implications of utilizing force-based methods as defined by the 1997 UBC for performance specification, and comparing the results to those obtained with a direct displacement-based method. It will be shown that the displacement-based design method achieves reliable designs while still retaining simplicity in the design process.

EXAMINATION OF UBC SEISMIC PROVISIONS APPLIED TO STRUCTURAL WALLS IN THE CONTEXT OF PERFORMANCE-BASED DESIGN

Overview of Performance Criteria

The 1997 UBC seismic provisions provide significant improvements over previous editions of the code for the design of structural wall buildings. For the first time, concrete compression strain limits are utilized directly in the design process. As stated in the code, a limit of $\varepsilon_c = 0.015$ is placed on the extreme fiber compression strain. In addition to the strain-based damage limit, interstory drift ratios are limited to a value of $\theta = 0.02$ or 0.025 depending on the period of the structure. Since there are two distinct deformation limits, it is of interest to determine when one limit state governs over the other. To accomplish this, the concrete compression strain limit is converted into a drift ratio limit as shown in the following.

The plastic rotation, θ_p, of a wall can be expressed in the following:

$$\theta_p = (\phi_u - \phi_y)L_p \quad (1)$$

where ϕ_u = ultimate curvature; ϕ_y = yield curvature; and L_p = plastic hinge length.

The elastic displacement profile, Δ_e, is given as a function of the distance up the wall, h_i, as shown in (2). Differentiation of (2) results in the elastic rotation θ_e, as a function of wall height, as shown in (3). The largest elastic rotation is at the top story, and substituting the total wall height, h_w, for h_i results in (4). The total interstory drift ratio, θ, is then the sum of (1) and (4) as shown in (5).

$$\Delta_e = \frac{\phi_y h_i^2}{3}\left(1.5 - \frac{h_i}{2h_w}\right) \quad (2)$$

$$\theta_e = \frac{d\Delta}{dh_i} = \phi_y h_i - \frac{\phi_y h_i^2}{2h_w} \quad (3)$$

$$\theta_{et} = \frac{\phi_y h_w}{2} \quad (4)$$

$$\theta = (\phi_u - \phi_y)L_p + \frac{\phi_y h_w}{2} \quad (5)$$

The UBC code utilizes (6) for estimation of yield curvature and (7) for plastic hinge length. It is noted that more reliable expressions are available elsewhere, however, for the purposes

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of this paper the UBC recommendations were followed exactly such that a meaningful assessment is possible. Assuming compatibility of deformation along the member cross section, the ultimate curvature can be expressed as a function of the concrete compression strain and neutral axis depth as in (8). Substituting (6)–(8) into (5) and introducing the variable A_r as the wall aspect ratio, given by (9), results in an expression for the drift ratio as a function of concrete compression strain, aspect ratio, and neutral axis depth [(10)]

$$\phi_y = 0.003/l_w \tag{6}$$

$$L_p = l_w/2 \tag{7}$$

$$\phi_u = \varepsilon_{cu}/c_u = \varepsilon_{cu}/\alpha l_w \tag{8}$$

$$A_r = h_w/l_w \tag{9}$$

$$\theta = \frac{1}{2}\left(\frac{\varepsilon_{cu}}{\alpha} - 0.003\right) + 0.0015A_r \tag{10}$$

$$\alpha = \frac{c_u}{l_w} \tag{11}$$

In (10), the UBC concrete compression strain limit of ε_{cu} = 0.015 can be inserted to obtain an expression for drift ratio as a function of aspect ratio, A_r, and neutral axis depth/wall length ratio, α. Previous research (Priestley and Kowalsky 1998) has shown that α is essentially constant for a given strain limit state defined by both concrete compression and steel tension strain limits. For example, if ε_{cu} = 0.018 and ε_s = 0.06, α = 0.20 (Priestley and Kowalsky 1998). However, the UBC code does not place any limit on steel tension strain. Therefore, consider two extreme conditions to determine the range of the variable α for a target concrete compression strain of ε_{cu} = 0.015. In one case, let the longitudinal steel ratio in a wall equal 0.0025, and the axial load ratio equal 0. In the other, the longitudinal steel ratios equals 0.025 and the axial load ratio is 0.10. Conducting moment curvature analysis on the wall section with a computer program developed by King et al. (1986), which utilizes the Mander et al. (1988) constitutive relation, results in α = 0.33 at a concrete compression strain of 0.015 (steel tension strain of 0.03) for the wall with 2.5% steel. The wall with 0.25% steel cannot achieve a concrete compression strain of 0.015 as the steel strain is well beyond the useable range (0.16) at a compression strain of only 0.008. Increasing the axial load ratio to 0.05 while keeping the steel ratio at 0.0025 results in a value of α = 0.11 at a concrete compression strain of 0.015 (the steel tension strain is 0.12). Considering the steel tension strains developed in these two extreme examples, it can be argued that a suitable range for α would be from 0.10 to 0.30. Substituting ε_{cu} = 0.015, and α = 0.10, α = 0.30 into (10) results in (12) and (13), respectively, which represents the drift ratio limits based on the prescribed concrete compression strain limit.

$$\theta = 0.0735 + 0.0015A_r \quad (\alpha = 0.10) \tag{12}$$

$$\theta = 0.0235 + 0.0015A_r \quad (\alpha = 0.30) \tag{13}$$

Fig. 1 represents a plot of (12) and (13), along with the absolute drift ratio limits of 0.020 and 0.025 and the elastic drift of the wall. From this figure, it is apparent that the structural capacity limit state as defined by a concrete compression strain of 0.015 will rarely govern the design of a structural wall with the exception of walls of aspect ratio less than one. Given that absolute drift ratio limits will usually govern design, and that for slender walls the allowable drift will be accommodated largely by elastic response as shown in Fig. 1, it is clear that the ductility demand at the drift ratio limits will be highly variable, and in some cases less than one. Consider the following scenario.

If α = 0.10, the curvature ductility as calculated by (14) is 50 for a concrete compression strain of 0.015. If α = 0.30, the curvature ductility from (15) is 16.7 for a concrete compression strain of 0.015. The displacement ductility at an effective height of 2/3 the total wall height can be shown to be related to the curvature ductility through (16) (Priestley and Kowalsky 1998)

$$\mu_\phi = \frac{\phi_u}{\phi_y} = \frac{0.015/0.10l_w}{0.003/l_w} = 50 \tag{14}$$

$$\mu_\phi = \frac{\phi_u}{\phi_y} = \frac{0.015/0.30l_w}{0.003/l_w} = 16.7 \tag{15}$$

$$\mu_\Delta = 1 + 3(\mu_\phi - 1)\frac{3}{4A_r}\left(1 - \frac{3}{8A_r}\right) \tag{16}$$

Plotting (16) with μ_ϕ = 50 and μ_ϕ = 16.7 results in Fig. 2, which illustrates the variation in ductility demand for different wall configurations. However, since the drift ratio based on the concrete compression strain of 0.015 will rarely govern the design, displacement ductility demand must also be calculated based on the maximum allowable drift limits of θ_{max} = 0.02 and 0.025.

Displacement ductility demand based on a maximum allowable drift ratio is obtained by subtracting the top-story yield drift ratio from the total allowable drift ratio. The remaining drift ratio is then the plastic drift ratio that can be expressed as a plastic curvature. The curvature ductility factor can then be expressed as a function of the maximum allowable drift ratio and the aspect ratio as in (17). Substituting (17) into (16)

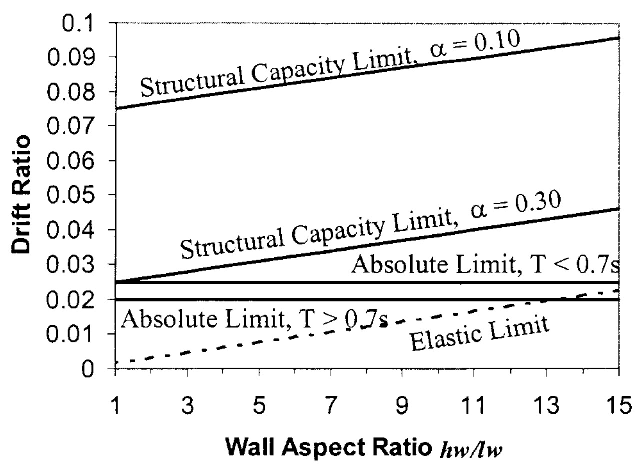

FIG. 1. Structural Capacity and Absolute Drift Limits

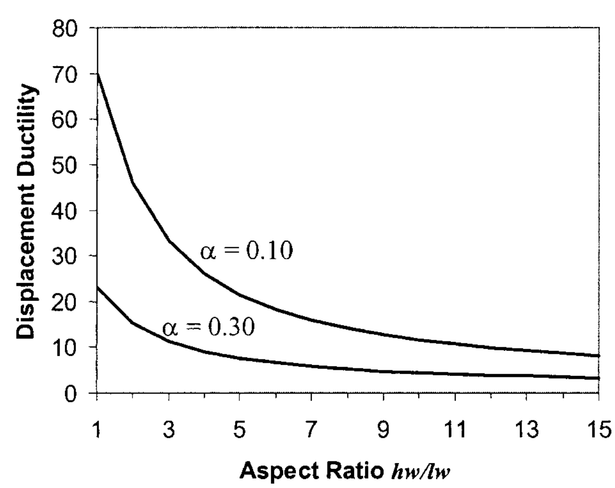

FIG. 2. Structural Capacity Ductility Levels

results in the displacement ductility factor based on the maximum allowable drift ratio [(18)]

$$\mu_\phi = 1 + 667(\theta_{max} - 0.0015A_r) \quad (17)$$

$$\mu_\Delta = 1 + 3[667(\theta_{max} - 0.0015A_r)]\,\frac{3}{4A_r}\left(1 - \frac{3}{8A_r}\right) \quad (18)$$

Eq. (18) can be plotted versus aspect ratio for the two values of maximum allowable drift ratio as shown in Fig. 3. Also shown in Fig. 3 is the assumed value of the force reduction factor for structural wall buildings multiplied by the UBC specified 0.7 factor. First, note that the range of ductility demand varies from a high of 20 for an aspect ratio of 1 to a value less than 1 for aspect ratios greater than 13. Given that the UBC code assumes a constant force reduction factor of 4.5, and an implied ductility demand of 3.15, it is clear that significant deviations between assumed and actual behavior will occur. The following summarizes the results of this portion of the paper:

1. The absolute drift limit employed by the UBC code for structural wall buildings will almost always govern the acceptance of a design as opposed to the concrete compression strain limit of 0.015.
2. The ductility levels based on the absolute drift limits as well as those based on the concrete compression strain limit vary greatly with aspect ratio, and a constant force reduction and hence ductility factor does not accurately represent the behavior of walls with an aspect ratio that deviates from a value of 8.

UBC Seismic Design

To understand the implication of this result on UBC seismic design, it is important to first discuss the basic steps that would be employed in a UBC design. The 1997 UBC design procedure utilizes force-based design concepts while performing a check on the deformations. In the equivalent lateral force procedure, the first step involves discretization of the structure and estimation of elastic period. The elastic period can be calculated with a simplified method or the more rigorous Rayleigh method. Once established, the design base shear is calculated along with minimum and maximum base shear values. The base shear is then distributed in plan to each wall based on wall stiffness and vertically within each wall according to wall height. A structural analysis is then performed to determine design base moments. In calculating the base shear force, a force reduction factor of $R = 4.5$ is utilized.

Acceptance of the resulting design is judged based on two

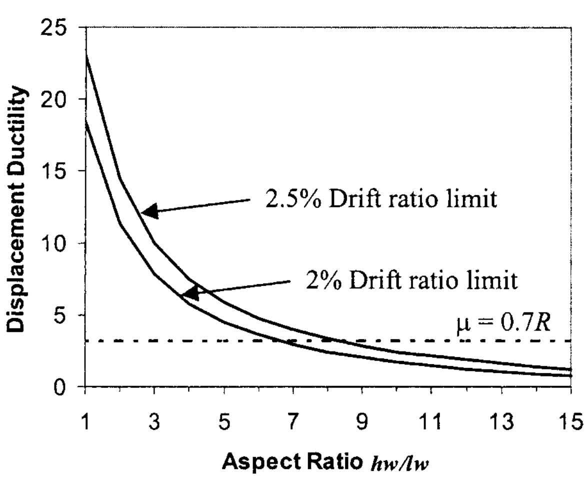

FIG. 3. Drift Limit Ductility Levels

deformation criteria, as previously mentioned. The first criterion requires that the compression strain in the extreme concrete fiber be less than 0.015 under the applied loading. The second criterion requires that the interstory drift ratio in any given story under the applied loading be less than 0.025 if the elastic period is less than 0.7 s, or less than 0.020 if the elastic period is greater than 0.7 s. As previously noted, the second criterion generally governs the design. Estimation of maximum deformation under the applied loading is performed by one of two methods.

1. The first method evaluates the maximum deformation by multiplying the elastic deflections by 0.7 R, where R is the assumed force reduction factor. The elastic deflections are obtained by applying the lateral force vector to a suitable analytical model that considers the effect of cracked sections or by conducting a dynamic analysis. The dynamic analysis procedure follows the traditional modal analysis method.
2. The second method utilizes dynamic inelastic time history analysis.

In this paper, both of these methods are utilized to determine acceptance of a series of structural wall buildings. For method 1, analysis is conducted using the same analytical model employed in estimating the fundamental period. As a result, cracked section stiffness is considered in the analysis. Elastic deflections for method 1 are obtained using the equivalent lateral force method. For method 2, the computer program Ruaumoko (Carr 1998) is utilized. Three separate analyses are performed under three different time histories that were generated to fit the design spectrum through the use of the computer program Simqke (Vanmarke 1976).

Based on the analysis results, if the largest calculated interstory drift ratio, which occurs at the top story, is higher than the limiting value of 0.02 or 0.025, then the structure would require modification though an increase in stiffness.

Direct Displacement-Based Design

In addition to investigating the UBC seismic design method, direct displacement-based design (DBD) is also applied to the same series of structural wall buildings. Given such a specific title as ‘‘displacement-based design’’ the variation in proposed approaches over the preceding 8 years is significant. Concentrating on design of new buildings only, the following observation can be made. The primary difference between the procedures known to the writer is the role that displacement, or deformation, plays in the design process.

In some cases, structural design is performed using current force-based approaches where base shear is obtained based on an estimate of the elastic period and application of force-reduction factors. Displacements under the forces obtained are then estimated using elastic analysis and deformation capacity is provided in the form of confinement reinforcement. Such an approach is an improvement over existing methods as deformation demand and deformation capacity are considered and transverse reinforcement sized accordingly (Moehle 1992; Wallace and Thomson 1995). A thorough review of displacement-based design methods was recently conducted by Priestley (2000).

A more direct interpretation of DBD for building structures was offered by Priestley (1993) and has been recently extended to the design of structural wall buildings (Priestley and Kowalsky 2000). The DBD approach utilizes as its starting point a target displacement that can be obtained with reference to damage-based (strain) or drift-based criteria. For a structural wall building, this consists of selection of a target displacement profile [Fig. 4(a)]. The DBD approach utilizes the sub-

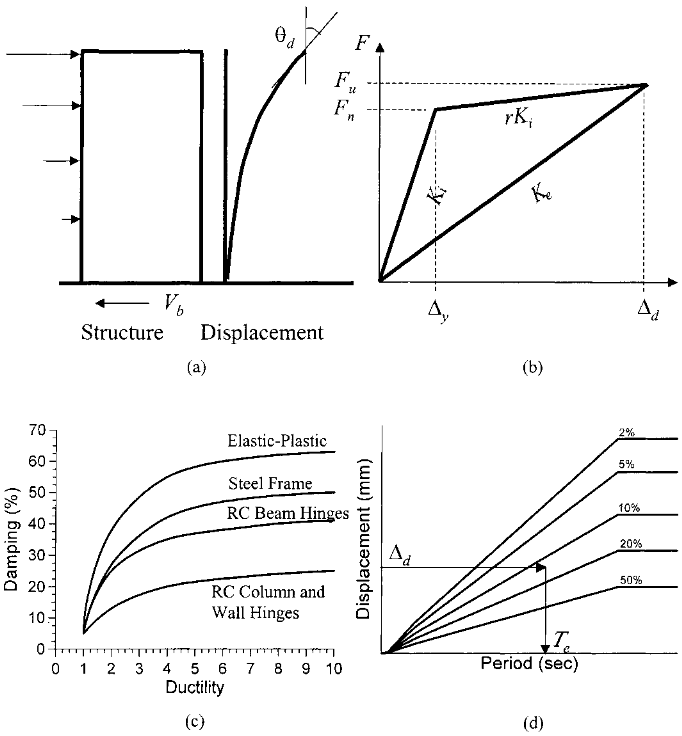

FIG. 4. Direct Displacement-Based Design Approach: (a) Establishing Target Displacement Pattern; (b) Effective Stiffness; (c) Effective Damping; (d) Effective Period

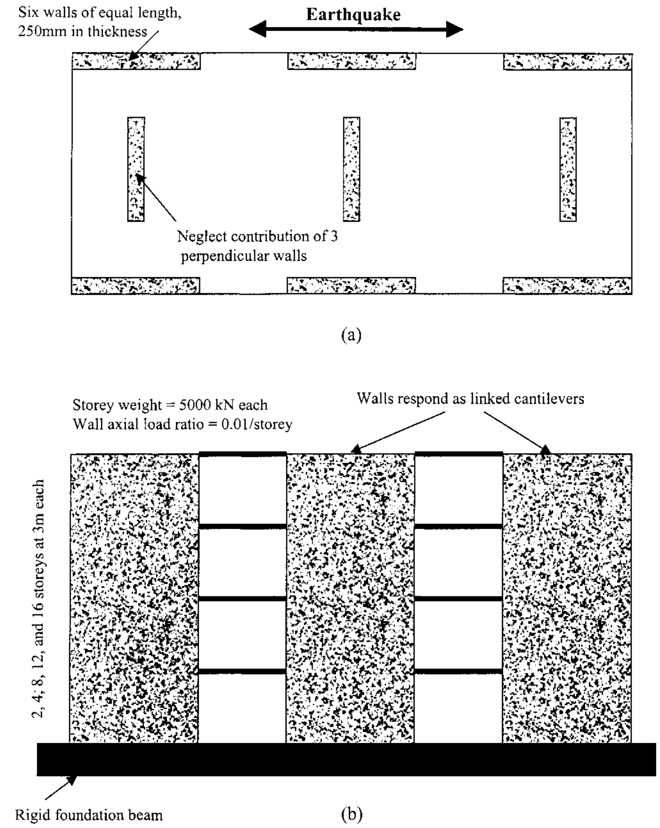

FIG. 5. Sample Building: (a) Plan View; (b) Elevation View

stitute structure approach (Gulkan and Sozen 1974; Shibata and Sozen 1976) to characterize the nonlinear behavior of an inelastic system with equivalent effective properties of effective stiffness [Fig. 4(b)] and effective damping. The effective damping characteristics for a particular hysteretic behavior can be obtained by employing Jacobsen's (1930) approach with the results expressed as a damping versus ductility relationship [Fig. 4(c)].

Once the target displacement profile is obtained and the yield displacement estimated, the ductility demand and hence effective damping is readily obtained [Fig. 4(c)]. The displacement response spectra, which represents the demand for the DBD approach is then entered with the target displacement. By reading across to the appropriate response curve given by value of effective damping previously calculated, the effective period at maximum response is obtained as shown in Fig. 4(d). The effective period is then expressed as an effective stiffness from consideration of a single-degree-of-freedom (SDOF) oscillator. The design base shear is then obtained by the product of the effective stiffness and target displacement [Fig. 4(b)]. Longitudinal reinforcement is then designed to resist the base shear demand, and confinement details provided to resist the target deformation demand. The reader is referred to Priestley and Kowalsky (2000) for more details.

DESIGN CASE STUDIES

To further explore the implications of using force-based methods to achieve deformation and hence performance control, a series of structural wall building design examples were considered. The basic structure is shown in Fig. 5. Six walls of equal length carry the lateral loads. In order to consider a reasonable range of wall aspect ratios, buildings of 2, 4, 8, 12, and 16 stories were considered. Each story was 3 m tall, and each wall resists 1% axial load ratio per story, and the story inertia weight is 5,000 kN/story. For each of the five different building heights, three different wall lengths were considered. In each case, the wall lengths are such that the required steel ratio falls between a minimum value of 0.25% and a maximum value of 2.5% for the UBC designed structures. The lower limit is required by the UBC code, while the upper limit was placed to avoid unrealistic designs as well as unrealistic comparisons. Walls were uniformly 250 mm thick. Concrete compression strength was selected as 27.5 MPa, while steel yield stress was selected as 400 MPa. Longitudinal bars were of 30 mm in diameter.

UBC provisions were followed exactly with the following interpretations: (1) cracked section stiffness for estimation of structural period and equivalent lateral force analysis was obtained in accordance with the recommendations of Paulay and Priestley (1992) (Eq. 19); (2) period calculations were performed using the Rayleigh method with the provision that the period would not exceed the simplified calculation method, given by (20), by more than 30%, as required by the UBC. In (20), the length of the wall is l_w, the width is b_w, and the height is h_w.

In accordance with the UBC provisions, the base shear is obtained with (21). In addition, a cap on the base shear is placed by (22), and the minimum base shear is given by (23). The buildings considered were designed for seismic zone 4, Sd type soil (C_v = $0.64N_v$; C_a = $0.44N_a$; Z = 0.4; N_a = N_v = 1). An importance factor of I = 1 was used, and a force reduction factor of R = 4.5, in accordance with UBC requirements for structural walls, was employed

$$I_{cr} = \left(\frac{100}{f_y} + \frac{P_u}{f'_c A_g}\right) I_g \tag{19}$$

$$T = \left[\frac{0.0743}{\sqrt{\sum l_w b_w \left(\frac{l_w}{h_w}\right)^2}}\right] h_w^{3/4} \tag{20}$$

$$V_b = \frac{C_v I}{RT} W \tag{21}$$

$$V_{b\max} = \frac{2.5C_aI}{R}W \tag{22}$$

$$V_{b\min} = \frac{0.8ZN_vI}{R}W \tag{23}$$

As part of the UBC design process, maximum deflections were calculated with both an equivalent lateral force analysis and dynamic inelastic time history analysis. It is recognized that the overwhelming majority of cases will involve deformation calculation with the equivalent lateral force method, and that dynamic inelastic time history analysis will rarely be performed. However, it was felt to be worthwhile to consider both since they are each acceptable methods for deformation calculation according to the UBC code. Furthermore, the dynamic inelastic time history analysis provides a point of comparison for the equivalent lateral force method.

In addition to designing the structures with the UBC force-based method and conducting two analyses on the resulting designs, the same buildings were designed with the DBD procedure previously described. The seismic input for DBD is expressed in the form of a displacement response spectra (DRS). The spectral values for the DRS were obtained by multiplying the ARS values by $T^2/4\pi^2$. A long-period modification was introduced for $T > 4$ s such that the DRS plateaus beyond 4 s. The effect of this modification on the UBC spectrum is minimal, and is shown in Fig. 6. Also shown in Fig. 6 is the design DRS for 5% damping. For displacement-based design, response spectra for damping values greater than 5% are required. These are obtained by utilizing the EuroCode 8 (CEC 1988) relation shown (24), which relates the spectral displacement response, Δ_ζ, at a damping value of ζ to the spectral displacement response at 5% damping, $\Delta_{5\%}$. In (24), ζ is expressed in percent

$$\Delta_\zeta = \Delta_{5\%}\left(\frac{7}{2+\zeta}\right)^{1/2} \tag{24}$$

The base shear for displacement-based design can be expressed by (25) (Kowalsky 2000) if a linear displacement response spectrum is assumed. From Fig. 6(b), it is clear that a linear assumption is reasonable, as the only nonlinearity occurs in the extreme-short-period range.

$$V_b = \frac{4\pi^2 m_{\text{eff}}}{\Delta_{\text{sys}}}\frac{\Delta_c^2}{T_c^2}\frac{7}{2+\zeta_{\text{eff}}} \tag{25}$$

In (25) the response spectrum shape is characterized by the variables T_c and Δ_c as shown in Fig. 6(b). The variable m_{eff} represents the effective mass and is given by (26) where Δ_i represents the target displacement profile as a function of story number i, m_i represents the mass of each story, and ns is the number of stories. The variable Δ_{sys} represents the system target displacement and is given by (27), while the variable ζ_{eff} represents the percentage of equivalent viscous damping for the building and is given by (28) (Kowalsky et al. 1995) where μ_Δ is the displacement ductility demand. Eq. (28) is based on the Takeda (1970) degrading stiffness hysteretic response and includes a 5% viscous damping component and the effective damping component based on the hysteretic energy dissipation.

For buildings with equal length walls, the effective damping of the building is the effective damping of any given wall, which is the case for the sample structure discussed here. For buildings with unequal wall lengths, the ductility and hence damping will vary for each wall. As a result, the effective damping for the building is obtained by combining the individual wall effective damping values in proportion to the work done by each wall.

$$m_{\text{eff}} = \sum_{i=1}^{ns}\frac{\Delta_i}{\Delta_{\text{sys}}}m_i \tag{26}$$

$$\Delta_{\text{sys}} = \frac{\sum_{i=1}^{ns} m_i\Delta_i^2}{\sum_{i=1}^{ns} m_i\Delta_i} \tag{27}$$

$$\zeta_{\text{eff}} = 100\left(0.05 + \frac{1 - \frac{0.95}{\sqrt{\mu_\Delta}} - 0.05\sqrt{\mu_\Delta}}{\pi}\right) \tag{28}$$

The target displacement profile for the displacement-based design procedure is based on the UBC drift limit of $\theta_{\text{lim}} = 0.02$ or 0.025, depending on the fundamental period of vibration of the building. The displacement profile, Δ_i, is obtained by (29) where the elastic profile, Δe_i, is given by (30), and the plastic profile, Δp_i, is given by (31). In (30), ϕ_y is the wall yield curvature and is given by (32) Priestley and Kowalsky 1998) where ε_y is yield strain of the longitudinal reinforcement. In (31), θ_{et} is the top-story elastic rotation which is shown as (33). Also, the plastic hinge length, L_p, in (31) is given by the greater of the values of (34) (Paulay and Priestley 1992) where d_{bl} and f_y are the longitudinal bar diameter and yield stress, respectively. In this paper, buildings have equal wall lengths. If the wall lengths vary, the displacement profile for the building should be based on the profile for the longest wall

$$\Delta_i = \Delta e_i + \Delta p_i \tag{29}$$

$$\Delta e_i = \frac{\phi_y h_i^2}{3}\left(1.5 - \frac{h_i}{2h_w}\right) \tag{30}$$

$$\Delta p_i = (\theta_{\text{lim}} - \theta_{et})\left(h_i - \frac{L_p}{2}\right) \tag{31}$$

$$\phi_y = 2\varepsilon_y/l_w \tag{32}$$

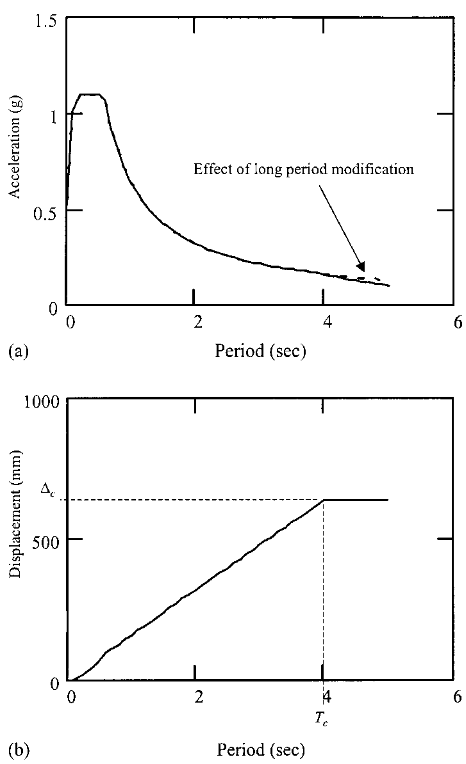

FIG. 6. Design Spectra: (a) Acceleration Response Spectrum for 5% Damping; (b) Displacement Response Spectrum for 5% Damping

$$\theta_{et} = \frac{\phi_y h_w}{2} \tag{33}$$

$$L_p = 0.2 l_w + 0.044 \left(\frac{2}{3} h_w\right) \tag{34a}$$

$$L_p = 0.08 \left(\frac{2}{3} h_w\right) + 0.022 f_y d_{bl} \tag{34b}$$

Dynamic inelastic time history analyses were also conducted on these designs in order to assess performance of the design procedure for deformation control. The 5% damped response spectra for the generated earthquakes are compared to the target design spectra in terms of acceleration, velocity, and displacement in Fig. 7. Although the overall agreement is good, there is some scatter. This should clearly translate into proportional scatter in terms of the dynamic analysis results. The analytical model in the time history analysis utilizes cracked section stiffness properties and the Takeda et al. (1970) degrading hysteretic model (second slope stiffness ratio of 0.05, unloading factor = 0.50, reloading factor = 0, and Emori unloading). Tangent stiffness Rayleigh damping in the amount of 5% in the first and third modes was assumed in the analysis. This is consistent with the 5% viscous damping assumed in design [first term in (28)] and should not be confused with the hysteretic damping component (second term) of (28).

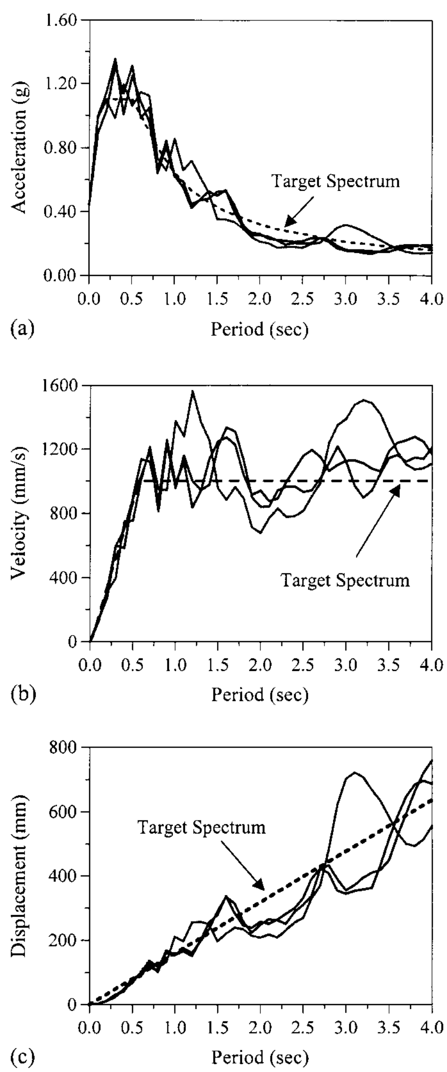

FIG. 7. Comparison of Spectra from Generated Earthquake and Target Spectrum: (a) Acceleration Response Spectrum, 5% Damping; (b) Velocity Response Spectrum, 5% Damping; (c) Displacement Response Spectrum, 5% Damping

Design and Analysis Results

Comparison of Design Base Shear

In this section, the design base shear force obtained with the UBC approach and the DBD procedure are compared for each of the building configurations. Fig. 8 illustrates the results. From Fig. 8(a), note that the UBC base shear remains constant regardless of wall aspect ratio for the two-story buildings. This occurs as a result of the maximum base shear expression [(22)] controlling the design. In contrast, the base shear from the displacement-based design procedure varies slightly with increasing aspect ratio recognizing that the more-slender walls will require slightly higher strengths to meet the same target drift ratio.

In the case of the four-story buildings [Fig. 8(b)], the UBC approach always required much higher base shear capacity than that required with displacement-based design. Also, the base shear is constant for the UBC approach as was the case for the two-story buildings, while a slight trend towards increasing strength requirements with increased flexibility is noted in the displacement-based design results.

The required base shear forces for the eight-story buildings are shown in Fig. 8(c). Note that unlike the two- and four-story buildings, the UBC base shear force now decreases as the stiffness decreases. This is in direct contrast to the displacement-based design results that indicate higher base shear force requirements as the stiffness decreases. This result is expected since displacement-based design considers inelastic response, and hence ductility directly while the UBC approach utilizes a fixed force-reduction factor. As the wall aspect ratio increases, displacement-based design recognizes the increased elastic deformation and hence reduced overall ductility demand. Meanwhile, the UBC force-based approach assumes that strength is independent of stiffness and that ductility, and hence force reduction is constant. As a result, elastic base shear is simply divided by the force reduction factor to obtain the

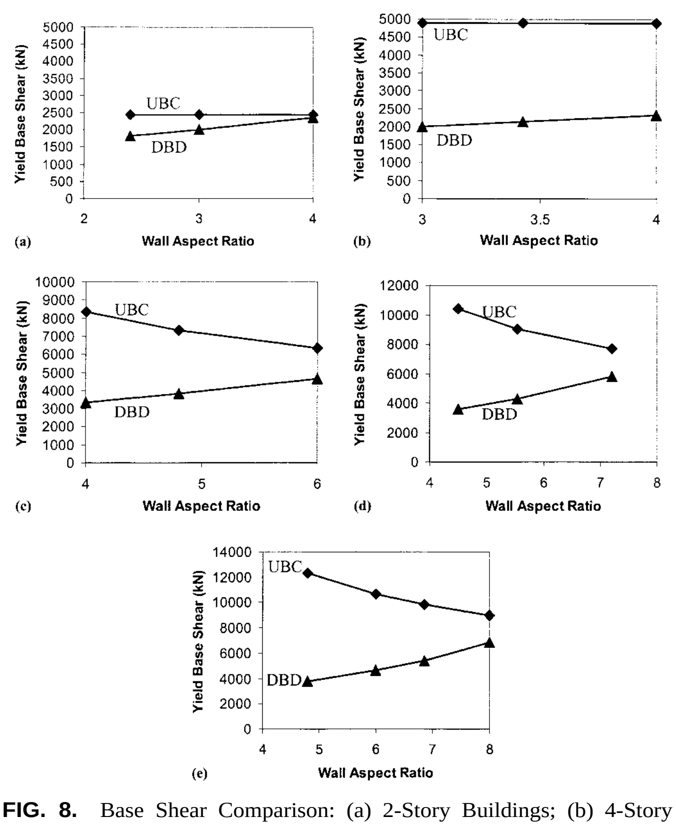

FIG. 8. Base Shear Comparison: (a) 2-Story Buildings; (b) 4-Story Buildings; (c) 8-Story Buildings; (d) 12-Story Buildings; (e) 16-Story Buildings

design base shear and since elastic base shear will reduce with decreased stiffness [see ARS of Fig. 5(a)], the result of Fig. 8(c) is expected. Similar results are apparent for the 12- and 16-story buildings in Figs. 8(d and e), respectively.

Comparison of Deformation Demands–UBC Analysis

In this section, the deformation demands as calculated using the UBC equivalent lateral force method are compared to the dynamic inelastic time history analysis. Results for the two-story buildings are shown in Fig. 9(a) where it is apparent that a significant disparity exists between the two analysis approaches. Of significance here is that all three designs would be deemed acceptable if dynamic inelastic time history analysis were conducted, however, only two are acceptable with the equivalent lateral force analysis method. In the case of the larger aspect ratio, the equivalent lateral force analysis would send the designer into an unnecessary loop of increasing the stiffness of the structure.

At the other extreme is the building with a wall aspect ratio of 2.4 where the equivalent lateral force method implies limited inelastic response with a maximum drift ratio of 0.006. In this case, the equivalent lateral force method provides a nonconservative result in addition to contradicting the assumptions made during the design of the structure. First, as noted in Fig. 9(a), the results of the time history analysis indicate that the maximum displacement, and hence damage would be much higher than expected from the equivalent lateral force procedure (although less than the allowable level). Second, the equivalent lateral force analysis indicates essentially elastic response while a force reduction factor of 4.5, which implies much greater damage, was assumed in design, as well as in the equivalent lateral force analysis. This result should be very troublesome to an astute engineer who would recognize that the assumptions made in the design (force reduction factor of 4.5) and the analysis (inelastic displacements = elastic displacements multiplied by 0.7 R) are invalidated by the results of the analysis themselves, which indicate essentially elastic response. Although this design would be acceptable as witnessed by the time history analysis, much confusion regarding the performance is prevalent, unless a more detailed analysis is performed which would rarely be the case.

Results for the 4-, 8-, 12-, and 16-story buildings are shown in Figs. 9(b–e). The same trends observed for the two-story buildings are apparent for the other buildings as well. However, for all of the 8-, 12-, and 16-story buildings, the equivalent lateral force analysis resulted in larger deformations than the dynamic inelastic time history analysis. Also, most of the equivalent lateral force analysis indicated maximum interstory drifts that exceed the allowable level, while the dynamic inelastic time history analysis indicated that all of the designs are satisfactory.

Target and Analysis Displacements—DBD Approach

The same structures were designed with displacement-based design where the target drift ratio was selected based on the cracked section period. In the case of the two- and four-story buildings, this resulted in a target drift ratio of 0.025. For all other buildings, the target drift ratio was selected as 0.020.

The maximum interstory drift ratios are shown versus aspect ratio in Fig. 10(a) for the two-story buildings. Unlike the force-based design procedure, displacement-based design provides three acceptable design results for three different wall lengths. Furthermore, in the displacement-based design approach, the designer has knowledge of the deformations with much greater accuracy than the UBC equivalent lateral force approach, while still retaining simplicity in the design process. Similar results for all building configurations are shown in Figs. 10(b–e).

An interesting observation can be made by analyzing the results of the 8-, 12-, and 16-story buildings. Recall from Fig. 9 that the UBC equivalent lateral force method would not allow a wall aspect ratio greater than 4, 4.5, and 4.75 for the

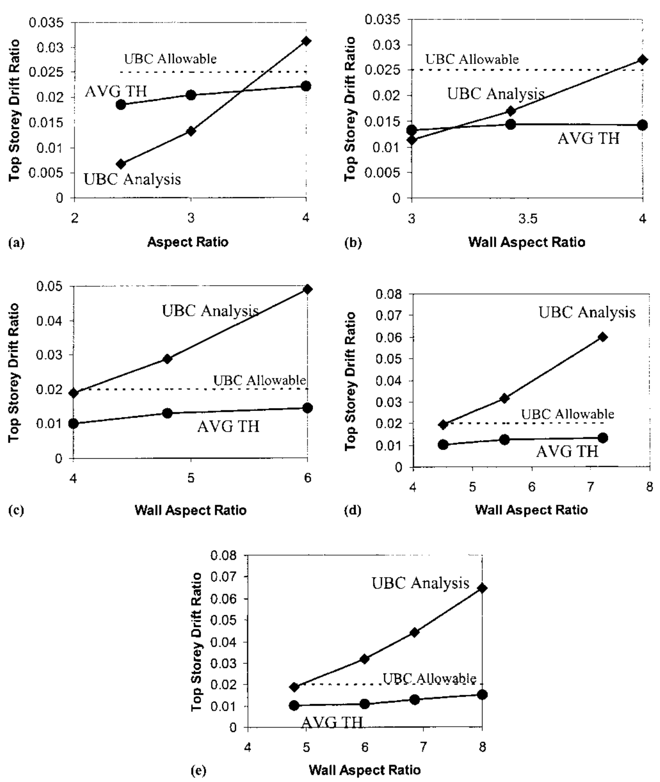

FIG. 9. Deformation Demands—UBC Analysis: (a) 2-Story Buildings; (b) 4-Story Buildings; (c) 8-Story Buildings; (d) 12-Story Buildings; (e) 16-Story Buildings

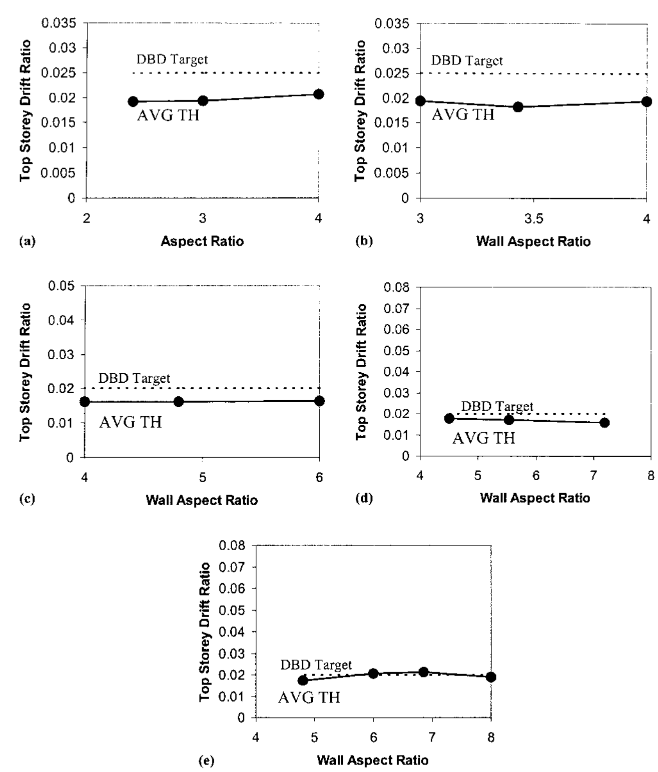

FIG. 10. Target and Analysis Displacement—DBD Approach: (a) 2-Story Buildings; (b) 4-Story Buildings; (c) 8-Story Buildings; (d) 12-Story Buildings; (e) 16-Story Buildings

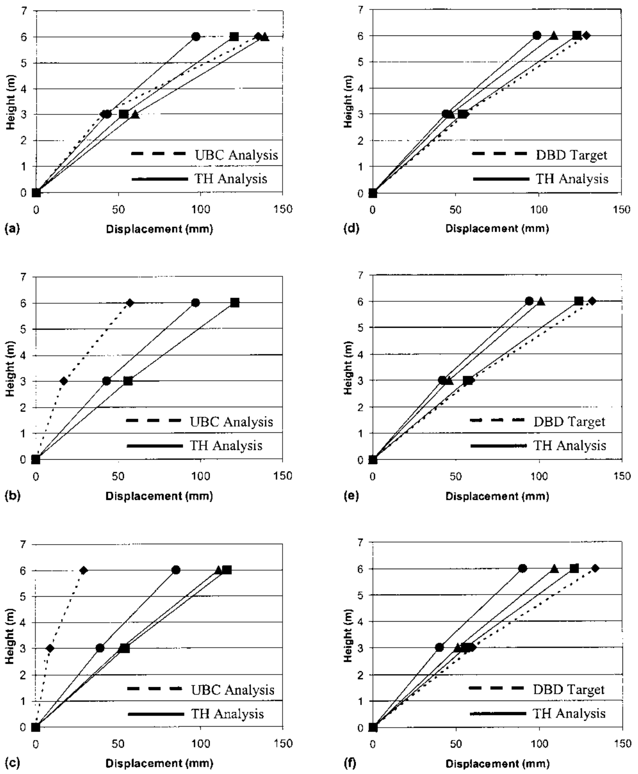

FIG. 11. Time-History Analysis Results for Two-Story Buildings: (a) UBC—1.5 m Wall Length; (b) UBC—2 m Wall Length; (c) UBC—2.5 m Wall Length; (d) DBD—1.5 m Wall Length; (e) DBD—2 m Wall Length; (f) DBD—2.5 m Wall Length

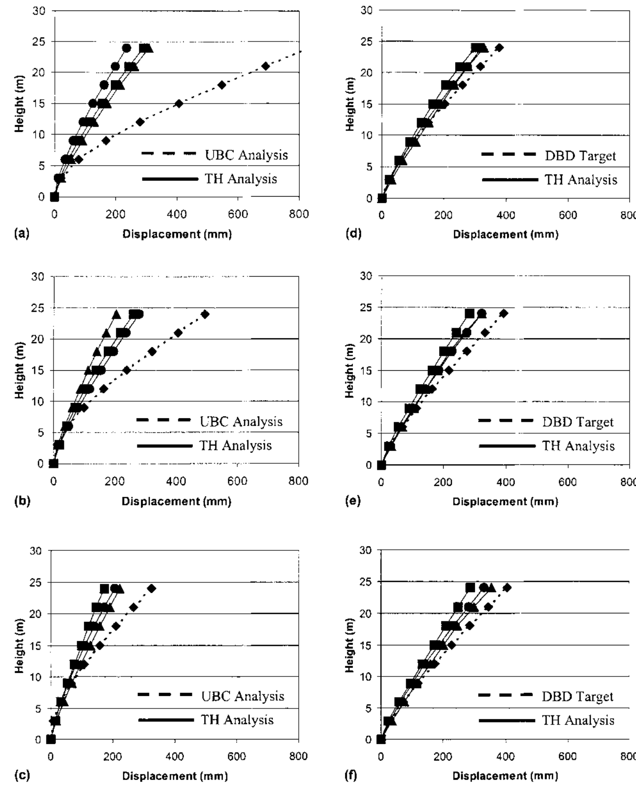

FIG. 13. Time-History Analysis Results for Eight-Story Buildings: (a) UBC—4 m Wall Length; (b) UBC—5 m Wall Length; (c) UBC—6 m Wall Length; (d) DBD—4 m Wall Length; (e) DBD—5 m Wall Length; (f) DBD—6 m Wall Length

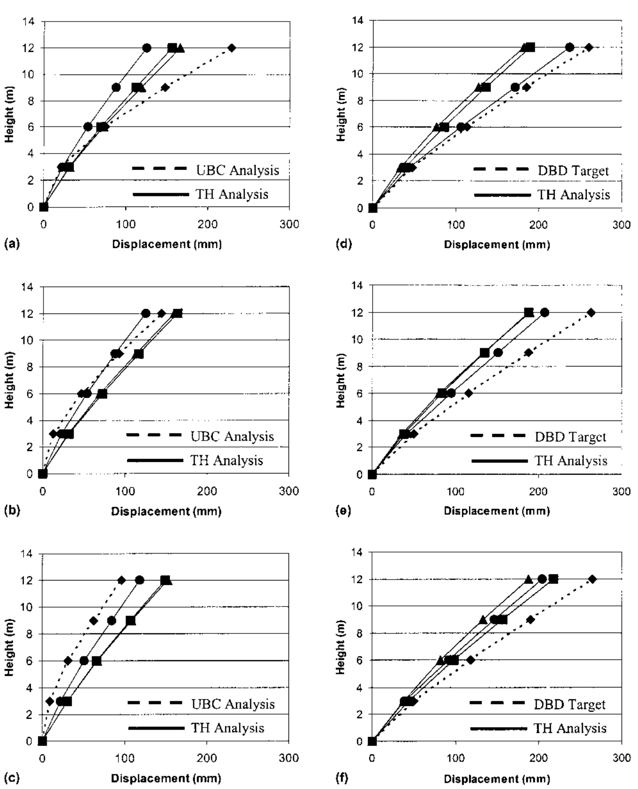

FIG. 12. Time-History Analysis Results for Four-Story Buildings: (a) UBC—3 m Wall Length; (b) UBC—3.5 m Wall Length; (c) UBC—4 m Wall Length; (d) DBD—3 m Wall Length; (e) DBD—3.5 m Wall Length; (f) DBD—4 m Wall Length

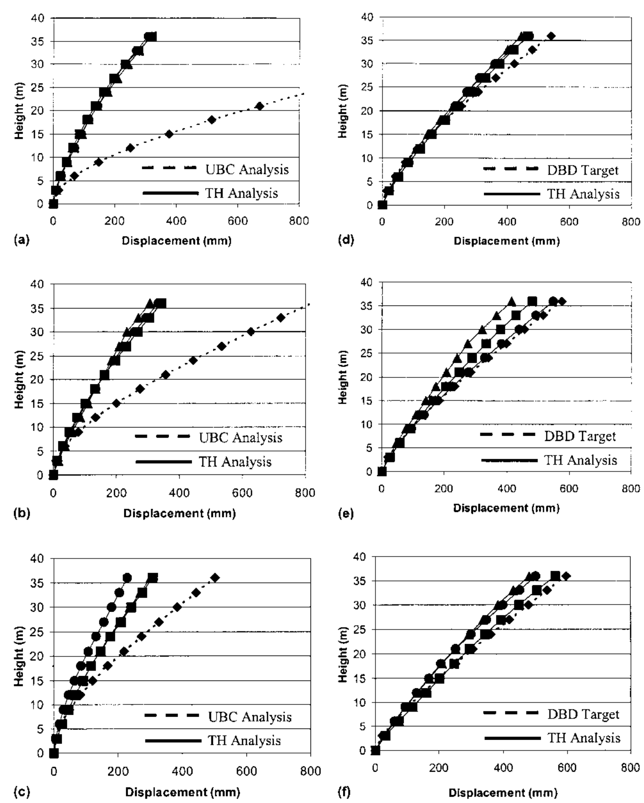

FIG. 14. Time-History Analysis Results for 12-Story Buildings: (a) UBC—5 m Wall Length; (b) UBC—6.5 m Wall Length; (c) UBC—8 m Wall Length; (d) DBD—5 m Wall Length; (e) DBD—6.5 m Wall Length; (f) DBD—8 m Wall Length

8-, 12-, and 16-story buildings, respectively. However, when considering the DBD results, it is noted that much larger aspect ratios (smaller wall lengths) are acceptable while still meeting the drift criteria.

Displacement Envelopes

Displacement envelopes for all analysis results are shown in Figs. 11–15. Fig. 11 represents the results for the two-story buildings. Note that the results of the analysis on the UBC structure with a wall length of 1.5 m [Fig. 11(a)] indicates that the equivalent lateral force analysis provides acceptable agreement between expected and actual displacements. Increasing the wall length to 2 m has a dramatic effect on the results of the equivalent lateral force analysis [Fig. 11(b)], as they are now much less in magnitude than the results of the time history analysis. The trend is even more dramatic for the 2.5-m-long walls in Fig. 11(c). An interesting point to note is that increasing the stiffness of the walls by increasing the wall length has little effect on the results of the time history analysis while having a significant effect on the equivalent lateral force analysis.

The displacement envelopes as a function of wall height are shown in Fig. 12 for the four-story buildings. The same trends as observed for the two-story buildings are evident for the four-story buildings. As the wall stiffness is increased, the equivalent lateral force analysis results are effected significantly [Figs. 12(a–c)]. Also, in all three cases there is some

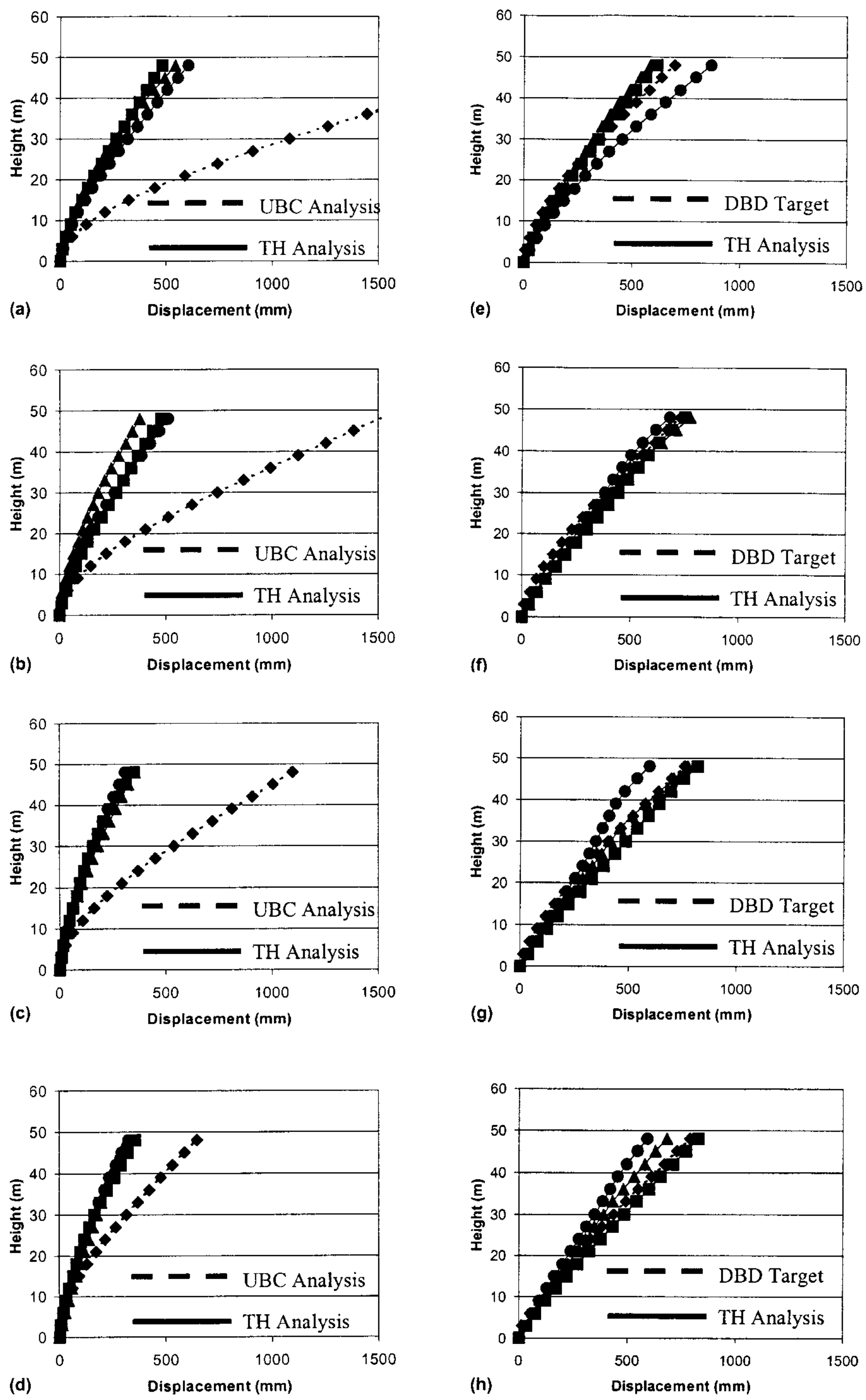

FIG. 15. Time-History Analysis Results for 16-Story Buildings: (a) UBC—6 m Wall Length; (b) UBC—7 m Wall Length; (c) UBC—8 m Wall Length; (d) UBC—10 m Wall Length; (e) DBD—6 m Wall Length; (f) DBD—7 m Wall Length; (g) DBD—8 m Wall Length; (h) DBD—10 m Wall Length

deviation between the time history analysis and the lateral force analysis in terms of magnitude and shape. The displacement-based design results are more consistent as shown in Figs. 12(d–f). Similar results are noted for the 8- and 12-story buildings in Figs. 13 and 14, respectively.

Results for the 16-story buildings are shown in Fig. 15. Note the very large difference between the equivalent lateral force method and the time history analysis in Figs. 15(a–d). The displacement-based design results provide good agreement as shown in Figs. 15(e–h). However, for the first time, the shape of the target displacement profile varied from that of the time history analysis, indicating that higher mode effects captured in the time history analysis should be explored in establishing the target displacement profile for design.

CONCLUSIONS

Given the current interest in performance-based earthquake engineering, the 1997 UBC seismic provisions for structural wall buildings were examined for their ability to specify and control damage in the design process. It was noted that the UBC places two distinct deformation limits for structural wall design: (1) a maximum extreme fiber concrete compression strain of 0.015 and (2) a maximum interstory drift ratio of 0.02 or 0.025, depending on the first fundamental period of the building.

These two limits were compared and it was concluded that the drift limit generally governs the design. It was further shown that displacement ductility levels at the allowable drift ratio limits very greatly, and for walls with large aspect ratios, the drift limit could be accommodated with essentially elastic response. A significant inconsistency was then noted as constant force reduction, and hence ductility factors are assumed in the UBC force-based design, which have the potential for introducing gross errors in deformation estimation for buildings whose actual ductility levels are different.

To assess the implications of utilizing a force-based design procedure to achieve the objectives of performance-based earthquake engineering, a series of designs and analyses were performed on structural wall buildings ranging from 2 to 16 stories. The buildings were designed using the UBC force-based method, as well as an alternative displacement-based design procedure. The following statements summarize the results:

1. Assessment of peak inelastic deformation through application of elastic analysis and displacement amplification factors can result in gross errors that will in turn lead the designer down an unnecessary path of stiffening or softening the structure.
2. Control of performance is not possible when the analysis method employed yields gross errors in deformation assessment.
3. The UBC design method, and force-based design in general assume that strength is independent of stiffness. As a result, when applying the UBC design approach with the equivalent lateral force method of analysis, there will be only one wall configuration that will achieve a preset deformation limit. In the displacement-based design method, it is recognized that strength and stiffness are dependent on each other and any wall length can be designed to reach a preestablished displacement target, within the bounds of what is practically possible with regards to reinforcement content.
4. In the UBC design procedure, the prescribed deformation limits are not consistent with the prescribed force reduction factors, and hence ductility factors. Furthermore, application of the equivalent lateral force method for analysis of the resulting design utilizes displacement amplification factors that are then invalidated as a result of the analysis itself. As a consequence, an astute engineer is left with an acceptance analysis that is based on inaccurate assumptions and a structure whose actual performance is unknown unless a more detailed analysis is performed.
5. For UBC force-based design, the required base shear capacity reduces with reduction in stiffness due to the use of constant force reduction factors and the reduced response acceleration. In contrast, displacement-based design recognizes that decreased stiffness results in larger elastic deformation and hence reduced ductility and effective damping for the same target drift ratio. As a result, the required base shear capacity for displacement-based design increases as the stiffness decreases.
6. The use of strain limits in the UBC in principal are valuable for control of damage. However, the drift ratio limits were shown to generally govern the design and are probably too restrictive. Furthermore, in order to meet the spirit of strain as a damage control limit state variable, it is essential that the design procedure employed be able to reliably control and predict inelastic deformation response.
7. For the cases considered, UBC designs are generally very conservative and will meet the specified limit states as defined by drift and strain limits based on the results of dynamic inelastic time history analysis. However, if the more common equivalent lateral force analysis method is utilized for acceptance, the perceived structural performance will in some cases be very different from reality and as a result actual structural performance will be largely unknown.
8. The force-based method could be improved by providing more rigorous guidelines for cracked section stiffness and force reduction factors that consider the actual structural behavior rather than simply using constant reduction factors and allowing the engineer to estimate stiffness.
9. A comparatively simple displacement-based design method could be utilized to achieve performance-based earthquake engineering where limit states are defined by reasonable drift ratio limits as well as damage based strain limits. In such an approach, the performance of the structures is well established during the design process without the need for further analysis and verification.

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