A TWO-LEVEL-PERFORMANCE-BASED DESIGN OF REINFORCED CONCRETE STRUCTURAL WALLS

Mehrdad Sasani[1]

Abstract

Performance-based design can be defined as a systematic method of designing structural systems to achieve predictable and desirable performance of both structural and non-structural elements. In order to ensure the desirable performance of buildings at different design levels, the strength, stiffness, and deformability of the structures should be reasonably proportioned. To achieve this proportionality, a performance-based design of reinforced concrete (RC) structural walls at the serviceability and life-safety levels is presented. It should be mentioned that, although the equations developed in this study, at the life safety level, are for structural walls, the general procedure is equally applicable to any kind of RC structural systems.

It is initially shown that, except for short period structures, increasing the strength of wall sections, by adding more flexural reinforcement, has negligible effects on the structural performance at the life-safety level, where the structural system undergoes significant in elastic behavior. Then, the following step by step procedure is used to design the structural system. 1) For the serviceability level earthquake, the structural system is designed to remain almost elastic. 2) The drift at this design level is checked to have negligible non-structural damage, and if needed, the stiffness of the structure is increased. 3) Utilizing a simplified method of analysis, the maximum displacement demand and capacity of the structure at the life-safety level are calculated and compared. This method, which is verified using the results of different experimental data, incorporates the effective moment of inertia of the sections, concrete strain concentration, and low cyclic fatigue. 4) Considering the calculated maximum displacement, non-structural elements are checked to prevent their failure, and if needed, the stiffness is increased. 5) The base shear is checked to prevent brittle shear failure. The paper concludes by a simple design example and comparing the results with those based on applying current seismic code provisions it is shown that these provisions overestimate the wall displacement capacity by a factor of about two.

[1] Ph.D. Student, Dept. of Civil Engineering, 721 Davis Hall, University of California, Berkeley, CA 94720-1710

Introduction

A performance-based design can be defined as a systematic method of designing structural systems to achieve a predictable and desirable performance of both structural and non-structural elements under actions the system will probably undergo during its lifetime. In order to ensure the desirable performance of buildings at different design levels, the strength, stiffness, and deformability of the structures should be reasonably proportioned. To achieve this goal, it should be clearly understood which one of these structural characteristics primarily governs the design at different performance levels. For this purpose, examining strength and deformability of the structure at the life safety level, i.e. under strong ground motion, is a reasonable starting point.

In 1981, based on different small-scale multi-story reinforced concrete (RC) structures tested on a shake table, Sozen wrote the following: "The *strongest conclusion is ...in relation to drift control the strong wall did not do perceptibly better than the weak wall ...Flexurally, the weak wall had approximately one-fourth the strength of the strong wall. ...This review suggests strongly that it is time to rehabilitate drift as the pivotal criterion for earthquake-resistant design. ... What else is the usual structure to do other than not move beyond a certain lateral displacement?"* But Sozen added *"much of our intellectual armament is aimed at producing numbers in relation to problems dominated by flexural or rotational ductility though the ductility problem in the field is dominated by shear and bond failures which do not lend themselves to analysis following from first principles."* Perhaps, this was the reason that Sozen did not attempt to develop a design method based on drift.

This paper begins by reviewing previous studies conducted on the seismic performance of RC structural walls. It is shown that, except for short period structures, increasing the strength of wall sections by adding more flexural reinforcement has negligible effects on the structural performance at the life-safety level, where the structural systems undergo significant inelastic behavior. Since strength is not a governing parameter at the life-safety level, except for short period structures, the design procedure, outlined in this paper, starts with the design of structural walls at the serviceability level, utilizing current seismic code provisions. In this respect, a simple procedure for this level of design is discussed. After proportioning structural elements, they are designed at the life-safety level, using the design procedure that is outlined in this paper. At the end, a simple design example is provided and the results are compared with those of UBC (1997).

It should be noted that in this study, the pulse-type ground motions, or the near source effects, are not included. Also it is assumed that the inelastic behavior of structural walls is limited to the inelastic flexural response at the vicinity of the base, i.e. the shear deformation is not included and in the case of having varied flexural strength along the height, it is so that no other plastic zone will be developed, except at the bottom of the walls. It should also be mentioned that because the equal displacement rule does not hold for short period structures, the design procedure is not directly applicable to those structures, unless proper modifications are made.

Performance of RC Structural Walls under Severe Ground Motions

In this section, the performance of RC structural walls under severe ground motion is reviewed to examine if the strength of structural systems is a governing parameter at the life safety level.

Flexural Behavior of RC Structural Walls

Bernoulli's hypothesis of plane section remaining plane in flexure is not applicable to structural walls within the hinging region. In Figure 1, the schematic measured strain distribution at the base is compared to a linear one; the calculated compressive strain based on Bernoulli's hypothesis is considerably less than the measured strain. Based on more than twenty tests on isolated structural walls under lateral loads, with or without the presence of vertical loads, Oesterle (1986) discussed the radially "fanned" crack pattern as a basis for developing a compatibility relationship instead of the assumption of the linear distribution of strain along the section (Figure 1). He relates the accumulated tensile strain over the length, L_T, to that of the compressive strain over a relatively short length, L_C, which leads to concrete strain concentration.

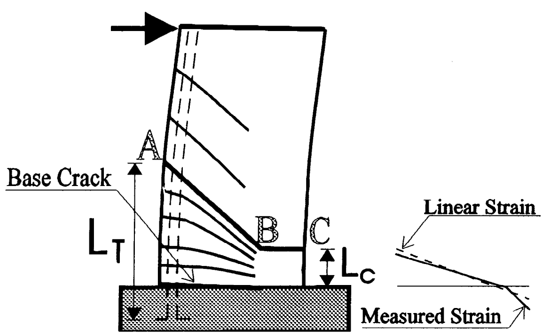

Figure 1. Fanned radially-cracked region at the bottom of a structural wall and schematic strain distribution at the base.

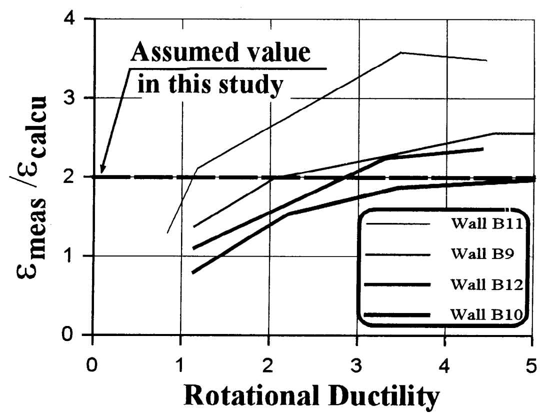

Figure 2. Ratio of measured to calculated (linear strain distribution) compressive strain of concrete versus rotational ductility.

Figure 2 shows the ratio or measured, ε_{meas}, to calculated, ε_{calcu}, (plane sections remain plane) maximum compressive strain versus rotational ductility for four structural walls with boundary elements, termed barbell walls, (Oesterle, 1986). The boundary element reinforcement ratios for these walls are between 0.02 and 0.037. Walls B11 and B12 are under lateral loads only while walls B9 and B10 also have axial loads. The concrete strength in wall B11 (53.7 MPa) is considerably higher than that of the other walls. Based on Figure 2, in this study, it is assumed that the maximum compressive concrete strain at the base of structural walls is twice as much as the result of a section analysis based on a linear distribution of strain.

Nonlinear Dynamic Analysis of RC Structural Walls

In this section a brief review of studies (Sasani and Anderson, 1996 and Sasani, 1997b) conducted on the seismic performance of RC structural walls is presented. Walls of 5, 10, and 15-stories, with 3.6 m story height, have been studied. The 5 and 10-story walls have uniformly reinforced rectangular cross sections with vertical reinforcement ratios of 0.0025, and 0.0075, called low and high reinforcement ratios, respectively. The 15-story wall has confined boundary elements with vertical reinforcement ratios of 0.01, low reinforcement ratio, and 0.03, high reinforcement ratio. The axial load is assumed to give a compressive stress at the base level equal to $0.05f'_c$ and $0.10\ f'_c$ for the 5 and 10-story walls, respectively. The compressive stress on the boundary elements of the 15-story wall, carrying the entire vertical loads, is 0.30 f'_c. These cases of different reinforcement ratios are investigated for each wall by varying the horizontal masses, m_H, to be 1.0, 2.0, and 3.0 times the vertical masses, m_v. The low cycle fatigue, as proposed by Fajfar (1992) is included. The Rayleigh damping of 5% in the first two modes is assumed. In this study, neither P-Δ effects nor the shear deformation effects are considered.

Nonlinear dynamic analyses of the above mentioned structural walls are conducted, using idealized elastic-perfectly plastic moment-curvature relationships for different sections. In calculating the moment-curvature relationships, the concrete strain concentration, as discussed above, is considered. Two different sets of accelerograms, each consists of sixteen records with PGA>0.2g and PGV>0.2 m/s, are used for the nonlinear dynamic analyses of structural systems and the displacement demand in each case is calculated. The first set of earthquake records is obtained from rock sites, whereas the second set of records is from alluvium sites. The walls are modeled as multi-degree of freedom, MDOF, systems with lumped mass at each floor level. Using the idealized moment-curvature relationships for different sections, assuming an inverted triangular distribution for the lateral load, and using the localized plastic hinge length, L_p, at the base of the structural walls, the displacement capacity at the top of each wall is calculated (the method of calculation is outlined in the following sections). Figure 3 shows the ratio between drift demand D_d and drift capacity, D_c, for all the cases studied.

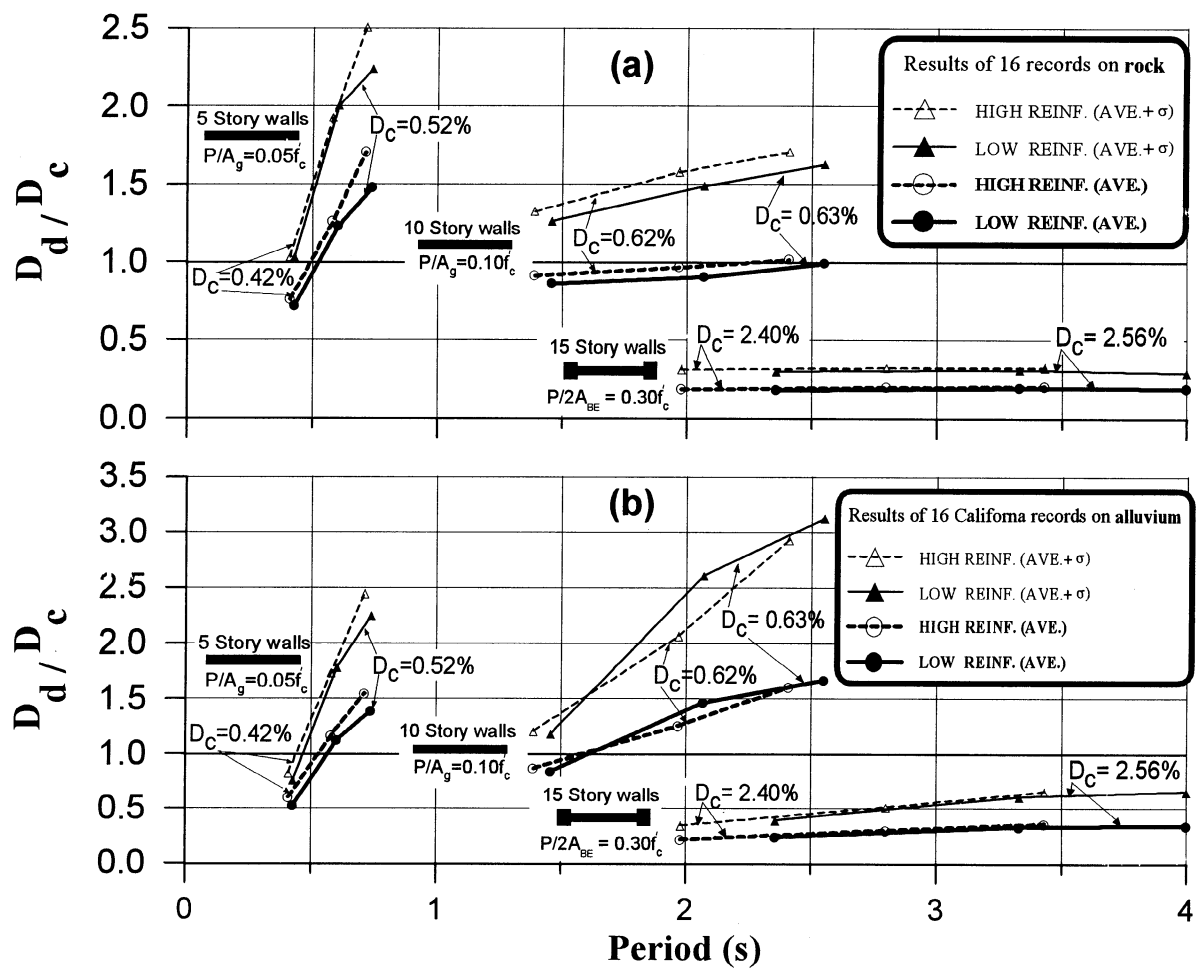

Figure. 3. Comparison between ratios of drift demand, D_d, and drift capacity, D_c, for low and high amounts of flexural reinforcement, (a) records on rock sites, (b) records on alluvium sites.

The sixteen accelerograms recorded on rock sites have $(PGA)_{ave}$=0.47g, $(PGV)_{ave}$=0.46 m/s, and $(PGD)_{ave}$=0.11 m. The records on alluvium sites have $(PGA)_{ave}$=0.38g, $(PGV)_{ave}$=0.44 m/s, and $(PGD)_{ave}$=0.17 m. As shown in Figure 3a (records on the rock sites) in all the cases, D_d/D_c for lightly reinforced sections are less than those for the more heavily reinforced sections. As shown in Figure 3b (records on the alluvium sites) for 5-story walls, in all the cases D_d/D_c for lightly reinforced sections are less than those for the more heavily reinforced sections. Although for 10 and 15-story walls D_d/D_c for lightly reinforced sections are not always less than those for the more heavily reinforced sections, except for mid-points of curves corresponding the 10-story buildings, D_d/D_c is almost the same for lightly and heavily reinforced sections.

It is important to note that, even for the 5-story walls with a period of 0.4s (the shortest period considered in this study) no improvement in performance is obtained with three-fold increase of the flexural reinforcement. For shorter period walls, because the equal displacement rule does not hold, this would not be true. Based on the above comparison, it is concluded that under random-types of severe ground motions, except for short period structures, strength is not a main concern in design and deformability of structural systems control the design.

Experimental Verification

Although the details of analysis are not provided in this paper, different comparison is made with the test results of four structural walls, tested under monotonic and also cyclic displacement, Table 1. As shown, there are good agreements between the measured and calculated values of moment of inertia at the first yield, the maximum displacement and also displacement ductility values. Again the comparison is discussed in detail by Sasani (1997b).

TABLE 1. Comparison between the experimental and the calculated results for the isolated structural walls previously tested.

Specimen - monotonic loading	Reference No.	I_y/I_g		Δ (m) or Θ (100*rad)			Displacement ductility		
		Meas.	Calcu.	Meas.	Calcu. Case 1[a]	Calcu. Case 2[a]	Meas. (monot.)	Meas. (cyclic.)	Calcu. (cyclic.)
PCA/B4	Oesterle, (1976)	0.12	0.13	0.226	0.253	0.253	18.8	8.3	8.7
PCA/SW2	Cardenas, (1973)	0.33	0.26	0.464[b]	0.530[b]	0.530[b]	N/A	N/A	N/A
UCB/SW3	Vallenas, (1979)	0.41	0.44	0.109	0.113	0.087	11.2	4.5	5.9
UCB/SW5	Vallenas, (1979)	0.43	0.50	0.050	0.064	0.057	7.7	4.2	4.6

[a] Cases 1 and 2 are based on two different eqs., to find the maximum usable concrete strain, (Sasani, 1997b)
[b] These values are the rotations at 0.16 of total height for specimen PCA/SW2

Design at the Serviceability Level

At this level of design, since structures are considered. to remain almost elastic, the design procedure is similar to the static force procedure used in UBC (1997). For example, for soil profile type S_B, since in this study the near source effects are not included, the following equation can be used as the strength requirement for structures.

$$\frac{0.8\, Z_s I}{\Omega_s} W < V_s = \frac{Z_s I}{\Omega_s T} W < \frac{2.5\, Z_s I}{\Omega_s} W \qquad (1)$$

where V_s is the base shear at the serviceability level, Z_s and Ω_s are the seismic zone factor and the overstrength factor at the serviceability level, respectively, that need to be determined; T, W, and I are the fundamental period, the weight and the importance factor as defined in UBC. For other types of soil profile, similar equations can be developed.

After proportioning structural elements under different load combinations, the maximum displacement should be checked to satisfy the requirements for nonstructural elements. If these requirements are not satisfied, the stiffness of the system needs to increase (the period needs to decrease) to limit the lateral displacement to the desirable level. Although clear requirements to limit the nonstructural damage are still lacking, a maximum drift of 0.005, as defined in Vision 2000 (1995) could be considered to minimize the nonstructural damage.

Design at the Life-Safety Level

As discussed above, under severe ground motions, the deformability of the system is the main design parameter. Wallace (1995) has developed a procedure for a displacement-based design of RC structural walls. Below, a displacement-based design procedure is presented. The procedure differs from Wallace's procedure in (a) taking into account the concentration of concrete strain as a result of shear cracks, (b) considering the difference between the cyclic and the monotonic behavior of structures, (c) incorporating an alternative way of calculating the stiffness (period) of structural walls and (d) considering the fact that the top lateral displacement of RC structural walls is mainly controlled by the response of the first mode of vibration. It is assumed that considering (a) and (b) would reduce the problems related to shear and bond failure that Sozen (1981) has mentioned. In this design procedure, the displacement demand and capacity are calculated and compared to see if enough capacity is available or the design needs to be revised. Also to control nonstructural failure, the maximum drift is limited to 0.02 (Vision 2000, 1995).

Displacement Demand

Because of its simplicity, and to avoid an inelastic analysis, the equal displacement rule has been used in most of the current seismic codes. Considerable studies have been carried out to find better period-dependent and also site-dependent relationships between the responses of inelastic systems and those of the corresponding elastic systems. One of the most recent studies has been conducted by Miranda (1993). Based on statistical analyses of a large number of records on different sites, he suggests different relationships between force reduction factors and periods of SDOF systems for different levels of displacement ductility and for different sites. Miranda's study show that, for SDOF systems with periods larger than about 0.5 to 0.7s the equal displacement rule is a good approximation.

There are not as many studies of MDOF systems as there are of SDOF systems. In 1984 Shimazaki and Sozen conducted some dynamic tests on MDOF systems. Wallace (1996) has reviewed the results of the tests and showed that, if the fundamental period of the MDOF systems is greater than the characteristic period of the ground motion, the maximum displacement response of inelastic systems can be reasonably estimated by the maximum displacement response of the corresponding elastic systems. Wallace and Moehle (1992) have suggested using 1.5 times of elastic displacement response spectra, as the inelastic displacement at the top of structural walls.

Sasani and Anderson (1996) and Sasani (1997b) have studied the behavior of different structural walls under the two different sets of records; the average top displacements are shown in Figure 4. Also the elastic displacement response spectra, multiplied by 1.5, are shown, which give a good, essentially conservative estimation of the top displacement of the inelastic walls.

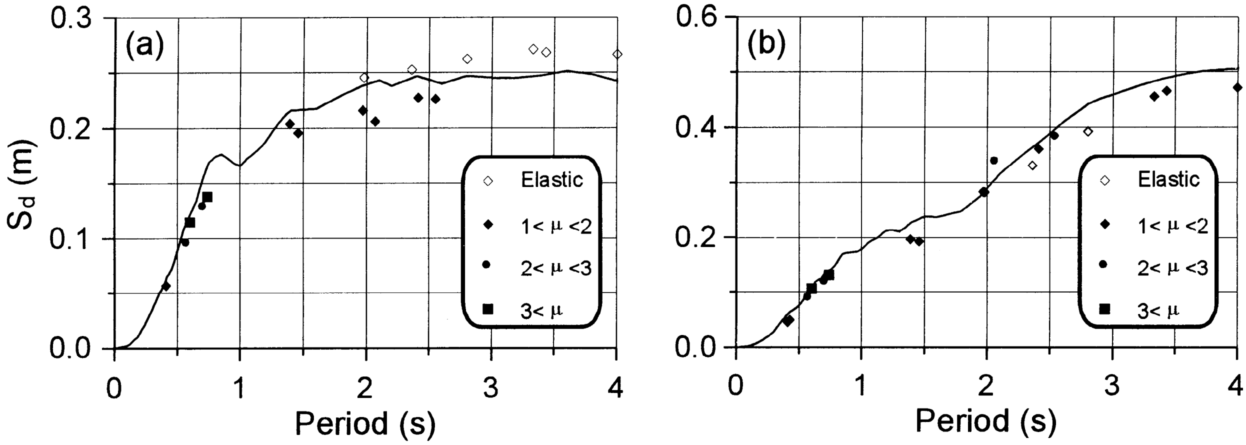

Figure 4. Comparison between the average maximum top inelastic displacement of structural walls and 1.5 times the corresponding elastic S_d. (a) records on rock sites (b) records on alluvium sites.

Elastic Displacement Response Spectra

In a simplified force-based design approach, acceleration response spectra are the basic tools used to find maximum forces. Similarly, in a displacement-based design, displacement response spectra provide convenient means to find maximum displacements. Although numerous studies have been conducted on the acceleration response spectra, the displacement response spectra have not attracted as many researchers. Therefore, in this study, the displacement response spectra are obtained from the acceleration response spectra.

For the soil profile type S_B, the Uniform Building Code (UBC, 1997) applies the following equation to determine the base shear, V, if there is no near source effects expected.

$$\frac{0.8ZI}{R}W < V = \frac{ZI}{RT}W < \frac{2.5ZI}{R}W \tag{2}$$

where, Z and R, are the seismic zone factor and the force reduction factor, respectively. Making use of the relation between the acceleration response spectra, S_a, and the displacement response spectra, S_d, and considering the fact that the lateral displacement of structural walls at the top is basically caused by the first mode of vibration and accordingly in calculating S_d only the modal mass in the first mode has to be considered, (Sasani, 1997a), S_d can be found, using the following equation.

$$S_d = S_a \cdot \frac{T^2}{(2\pi)^2} = 0.017\ g\ Z\ I\ T \qquad 0.4s < T < 1.25s \tag{3}$$

where g is the acceleration of gravity. Because of constant acceleration response spectra in the short (T<0.4s) and long (T>1.25s) period range, Equation. 3 needs to be modified in those regions. For T<0.4s and T>l.25s, T should be replaced by T^2, and the coefficient 0.017 should be replaced by 0.042 and 0.011, respectively. Note that perhaps the rapidly increasing S_d corresponding to long period structures need to be modified. For other types of soil profile, similar equations can be developed.

Equivalent Flexural Stiffness and the Fundamental Period of Vibration

Obviously, the fundamental period of a structures is the main parameter, aside from design spectra, to determine its maximum displacement. For this reason, it is important to have a reasonable estimation of the flexural stiffness. Unlike the traditional force-based design approach in which the fundamental period of vibration is underestimated to estimate forces conservatively, in a displacement-based design, the intention is the opposite to have a more conservative estimation of the maximum displacement.

Flexural Stiffness at the Base. At the lower parts of structural walls, the effects of cracked sections on flexural stiffness must be considered. In this study, the flexural stiffness of the lower part of a structural wall is assumed to be $EI_y = M_y/\Phi_y$, where M_y and Φ_y are the bending moment and the curvature at the first yield, respectively. Table 1 shows a comparison between this assumption and some test data.

Equivalent Flexural Stiffness. Paulay and Priestley (1992) suggest the following equation to find the equivalent moment of inertia of the wall cross section at the first yield, I_e,

$$I_e = \left(\frac{100}{f_y} + \frac{P_b}{A_g f'_c} \right) I_g \tag{4}$$

where P_b is the axial load at the base of the wall. A_g and I_g are the gross area and the gross moment of inertia of the section, respectively; and f_y is in MPa. In this equation the amount of reinforcement in the section is not taken into account, and cracked sections are assumed all over the height of the wall.

Under an inverted triangular distributed lateral load with a uniformly distributed gravity load over the height of a wall, and taking into account the effect of uncraked top portion of the structural wall, the following approximate equation is suggested for the equivalent flexural stiffness, EI_{eq}, over the entire height of the wall. Note that the effect of uncraked portion of a wall becomes more important as the amount of reinforcement in the section decreases. Compared to a more accurate analysis, Equation 5 predicts EI_{eq} of structural walls with rectangular and barbell cross sections within 15% error.

$$EI_{eq} = \left(\frac{65\left(1 + 0.65\rho_w + \bar{\rho}_{be}\right)}{f_y} + \frac{P_b}{A_g f'_c} \right) EI_g \tag{5}$$

where ρ_w is the percent of vertical reinforcement in the web$\bar{\rho}_{be}$ 2 $A_{be}\left(\rho_{be} - \rho_w\right) / A_g$, where ρ_{be} and A_{be} are the percent of reinforcement in the boundary element and the area of the boundary element, respectively. Note that all the reinforcement ratios are those at the base and f_y is in MPa.

Fundamental Period of Vibration Considering the fundamental period of vibration, T, of a cantilever structural system with uniformly distributed mass and stiffness and a total height of H, T of a structure with N structural walls as its lateral resisting system in the direction considered, can be found as:

$$T = 1.8\,H^2 \sqrt{\frac{\overline{m}}{\sum_{i=1}^{N} EI_{eq}}} \tag{6}$$

where $m = m_f/h$, m_f and h, being the average floor mass and the story height, respectively.

Displacement Capacity

The modes of failure that affect the ultimate behavior of structural walls are: boundary element crushing, bar fracture, inelastic bar buckling, instability of compressive concrete, web crushing, and sliding shear (Oesterle, 1985). In this study, it is assumed that the structural system is designed well enough to prevent shear failure. It is also assumed that walls are proportioned properly to prevent instability of the compressive zone. Therefore the displacement capacity of walls are controlled by concrete and steel strain. The inelastic reinforcement buckling is a complex problem and in this study it is assumed that if the transverse reinforcement is spaced at not more than six times diameter of longitudinal reinforcement, the bar buckling is controlled (Bertero and Collins, 1973). Although shear failure and deflection are not discussed in this study, shear cracks and their effects on strain distribution are considered. The effects of cyclic loading are also taken into account.

Assuming an inverted triangular distribution for the lateral load, the yield displacement at the top of a structural wall is $\Delta_y = \Phi_y H^2/3.6$. Therefore the displacement capacity at the top of the structural wall is given by the following equation:

$$\Delta_c^t = \frac{\Phi y \; H^2}{3.6} + (\Phi u - \Phi y)\, Lp \left(H - Lp/2 \right) \tag{7}$$

where Φ_y and Φ_u are yield and ultimate curvatures of the section. L_p is the plastic hinge length and H is the total height of the wall. For simplicity in this study, it is assumed that $L_P = L_w/2$, where L_w is the length of the wall in the plan. Other kinds of equations to calculate L_P of structural walls are discussed by Sasani (1997b). Assuming a bilinear moment-curvature relationship for the sections, yield curvature could be calculated as $\Phi_y = M_u/EI_{eq}$, where M_u is the ultimate moment capacity of the section. To calculate Φ_u, the maximum usable concrete strain is needed to carry out a section analysis. Also it should be mentioned that the section analysis needs to be corrected due to cyclic loading.

Maximum Usable Concrete Strain

As discussed above, the displacement capacity of structural walls is mainly controlled by concrete and steel strain. To investigate maximum usable concrete strain, ε_c^{max}, Kaar et al. (1976) have tested specimens, which have modeled the compression parts of structural walls. The following equation is obtained by linear regression of their test results.

$$\varepsilon_c^{max} = 0.004 + 0.002 \; f_{yh} \; \rho_{sh} \tag{8}$$

where ρ_{sh} is the volumetric ratio of confining hoops and f_{yh} is their yield stress in MPa.

Effects of Cyclic Loading

It has been observed that under cyclic loading ultimate curvature is somewhat less than that for monotonic loading. If one makes use of the relation between the cyclic displacement ductility, μ_Δ, and the monotonic ductility, μ'_Δ, as developed by Fajfar (1992) which is based on the Park-Ang damage equation, and also applies the average values for different parameters as Fajfar suggests and assumes a damage index equal to one, the following equation can be found.

$$\mu_\Delta = 3.33\left(\sqrt{1 + 0.6\,\mu'_\Delta} - 1\right) \quad (9)$$

Based on Equations 7 and 9, Figure 5 shows the relationship between Φ_u / Φ'_u and μ'_Φ, where Φ_u and Φ'_u are the cyclic and monotonic ultimate curvature, respectively, and μ'_Φ is the monotonic curvature ductility. In this figure, H/L_W varies between two and eight. Figure 5 also shows the line given by Equation 10, which is a reasonable approximation of different curves. Examining Equation 10, for small value of μ'_Φ, a more simplified assumption could be $\Phi_u / \Phi'_u = 0.7$.

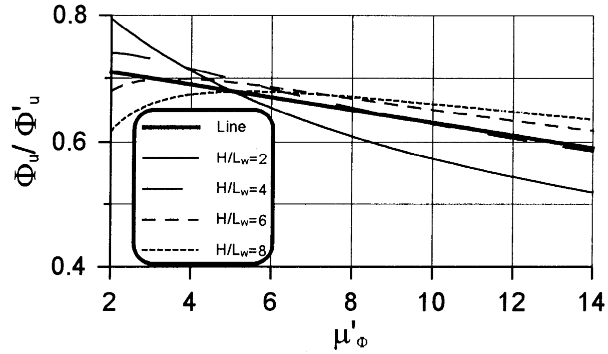

Figure 5. Relationship between Φ_u / Φ'_u and μ'_ϕ for different values of H/L_W.

$$\frac{\Phi_u}{\Phi'_u} = 0.7 - 0.01(\mu'_\Phi - 3) \quad (10)$$

Shear Design

Although the maximum top displacement and also the base moment (to a lesser degree) of structural walls under earthquakes could be reasonably calculated by an almost linear variation of lateral load along the height, this is not true for the base shear calculation. Based on experimental and also analytical analysis of RC structural walls, Eberhard and Sozen (1993) have suggested an equation to estimate the ultimate base shear, V_u, which can be expressed as below:

$$V_u = V_1 + 0.3\,ZW \quad (11)$$

where V_1 is the base shear capacity for the structure calculated by limit analysis assuming a triangular force distribution. In this study it is assumed that V_1=1.3/0.9*V_s, where the coefficient 1.3/0.9 is meant to consider the effects of bending moment reduction factor, 0.9, and also higher steel yield stress and strain hardening of reinforcement, 1.3.

Design Example

Using the outlined design procedure, a seven-story building is designed in this section. The plan of the building is shown in Figure 6. The story height is 3 m and a uniform gravity unit load of 6 kN/m^2 is assumed.

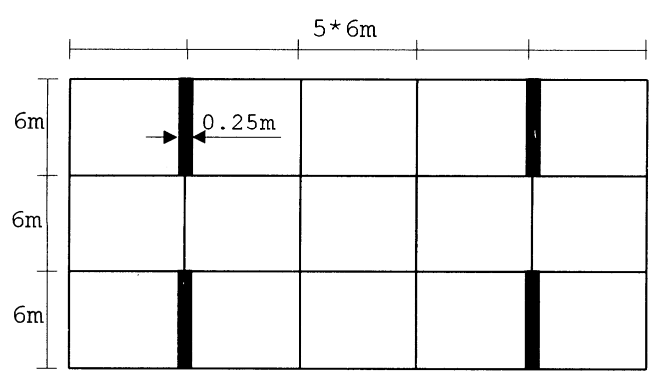

Figure 6. Plan of a seven-story building. Only the transverse lateral resisting system is shown.

Serviceability Level Design

1) Using equations (30-8) and (30-9) of UBC (1997), T=0.56s.

2) Assuming that Z_s=Z/3 and Ω_s=1.5, for Z=0.4, Equation 1 results in V_s=3.6 MN; the wall bending moment at the base is $M_{s,w}$=12.6MN-m

3) Knowing that 0.9 $P_{s,w}$=2.0 MN, where $P_{s,w}$ is the axial load of each wall at the base, the uniformly distributed vertical reinforcement ratio at the base is calculated as ρ_v=0.005(fy=410 Mpa & f'c=28 Mpa).

4) Replacing Z with Z_s in Equation 3, 1.5 times of S_d results in an average drift ratio of 0.0009<<0.005.

Life-Safety Level Design

1) Equation 5 leads to $EI_{eq} = 0.26EI_g$, ($\Phi_y = M_u EI_{eq} = 0.00045\ m^{-1}$). Using Equation 6, the fundamental period of the structure is calculated equal to 0.77s. (note that at serviceability level T=0.56s).

2) 1.5 times of S_d from Equation 3 results in a top displacement demand of 0.077m (D_d=0.0037<<0.02).

3) With no confinement, ε_c^{max} =0.004/2=0.002, a section analysis results in Φ'_u= 0.0020 m^{-1}. Since μ'_Φ is small, Φ_u=0.7 Φ'_u =0.0014 m^{-1}. Assuming $L_p=L_w/2$=3m, Equation 7 leads to Δ^t_c = 0.11m>0.076m.

4) Using Equation 11, the base shear V_u=7.9 MN (=0.35W), subsequently, the corresponding horizontal reinforcement ratio is the minimum value (ρ_n=0.0025).

Comparison with UBC (1997) Displacement-Based Design

Applying equation 21-9 from UBC (1997) to calculate the displacement capacity of the walls designed in the above example and assuming that ε_c^{max} =0.003, the top displacement capacity of 0.22m will be calculated which is twice as much as the capacity calculated in the above example. Obviously, the main reasons for this difference are not considering the concentration of concrete strain and also not including the effects of cyclic loading.If&,“““” =0.004 were used, the difference would be even higher. This tendency to over estimate the displacement capacity of structural walls in the code can also be examined by calculating the capacity of walls, which have been tested. For instance, UBC (1997) estimates the flexural displacement capacity of specimen UCB/SW5 at the same level that is given in Table 1 as 0.073m, which is more than twice as much as the measured one, under cyclic displacement, which is 0.035m. The monotonic flexural displacement capacity of this wall was measured at 0.050m. If the effect of having a larger period, because of smaller flexural stiffness, as discussed in this paper, is also considered, UBC (1997) provisions underestimates the displacement demand as well.

Conclusions

It is concluded that at the life safety level, except for short period structures and in the absence of pulse-type ground motion, flexural strength is not a governing parameter and the design at this level should be based on comparison between wall displacement capacity and demand. Therefore a design procedure is proposed starting with proportioning the structural elements at the serviceability level to satisfy the strength requirement of the structural system and also to limit the deformation to minimize the nonstructural damage. Then the design procedure continues by designing, checking, and detailing of the walls at the life safety level, incorporating the displacement-based design outlined in this paper. At the end the base shear is checked to prevent brittle shear failure of the structural walls. It is also shown that by not including the effect of concrete strain concentration caused by shear cracks and also ignoring the effects of cyclic loading on the wall displacement capacity, UBC (1997) design provisions can overestimate the displacement capacity of structural walls by more than a factor of two.

References

Bertero, V. V. and R. G. Collins. "Investigation of the Failure of the Olive View Stairtowers During the San Fernando Earthquake.", Report EERC 73-26, Univ. of California, Berkeley, Dec. 1973.

Eberhard, M.O. and M.A. Sozen. "Behavior-Based Method to Determine Design Shear in Earthquake-Resistant Walls." *Journal of the Structural Engineering ASCE,* 119 (Feb. 1993): 619-640.

Sozen, M. A. "Review of Earthquake Response of RC Buildings with a View to Drift Control." *State-of the- Art in Earthquake Engineering, 7thWCEE* (1981): 383-418.

Fajfar, P. "Equivalent Ductility Factors, Taking into Account Low-Cycle Fatigue." *Earthquake Engineering and Structural Dynamics.* 21(1992): 837-848.

Miranda, E. "Evaluation of Site-Dependent Inelastic Seismic Design Spectra." *Journal of the Structural Engineering, ASCE,* 119 (May 1993): 1319-1339.

Oesterle, R. G. "Inelastic Analysis for In-Plane Strength of Reinforced Concrete Shear Walls." Ph.D. dissertation, Northwestern Univ., Evanston, IL, 1986.

Paulay, T. and M. J. N. Priestley. *Seismic Design of Reinforced Concrete and Masonry Buildings.* John Wiley & Sons, 1992.

Sasani, M. and D.L. Anderson "Displacement-Based Design Versus Force-Based Design for Structural Walls." Paper no. 32. Proc. *11WCEE,* Acapulco, Mexico, 1996.

Sasani, M. "Discussion of Evaluation of UBC-94 Provisions for Seismic Design of RC Structural Walls by J. W. Wallace." Earthquake Spectra 13, no. 1 (1997a): 155-159.

Sasani, M. "Performance of RC Structural Walls under Severe Ground Motions." Submitted for review. (1997b).

Uniform Building Code. Vol. 2. International Conference of Building Officials, Whittier, CA., 1995.

Vision 2000. *Performance Based Seismic Engineering of Buildings.* SEAOC, 1995.

Wallace, J. W. "Seismic Design of RC Structural Walls. Part I: New Code Format." *Journal of the Structural Engineering, ASCE,* 121 (Jan. 1995): 75-85.

Wallace, J. W. "Evaluation of UBC-94 Provisions for Seismic Design of RC Structural Walls." *Earthquake Spectra* 12 (May 1996): 327-348.

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