7th Canadian Conference on Earthquake Engineering / Montreal / 1995
7ième Conférence canadienne sur le génie paraséismique / Montréal / 1995

The Seismic Performance of Different Strength Structural Walls

M. Sasani[1] and D.L. Anderson[2]

ABSTRACT

An investigation of seismic structural walls with different strengths, different cross sections and different periods has been carried out to compare the performance of these structural systems in terms of maximum concrete compressive strain or maximum steel strain . The results show that, except for short period structures, weaker structures perform as well if not better than stronger structures. An equation for calculating plastic hinge length has been interpreted. Considering the amount of reinforcement in the section and low-cycle fatigue, a definition for an idealized moment-curvature relationship has been suggested.

INTRODUCTION

To design a structure properly for seismic loads, one should identify the critical regions that govern the design and then detail them with care. As long as safety (ultimate) level design under strong ground motions is concerned, in most cases economical design of the structures forces the designer to accept inelastic behavior of the system. Consequently adequate inelastic performance of the critical regions of the structure is of primary concern for a well designed system. Thus, seismic design is different from gravity load design and one of the issues that comes up is what should be the basis for design, should it be based on forces or should control of deformations be the design criteria. This paper will attempt to show that a deformation based design criteria may produce structures with more uniform levels of safety than the more traditional force based designs.

Assume that the strength of a structural wall, with the same concrete cross section, is increased by 50% by adding more reinforcement, and that this has negligible effect on the stiffness of the section (this effect will be considered later). As is shown in Fig. 1, by assigning equal displacement ductility to both walls, the stronger wall is assumed to have higher displacement capacity by the same 50%. But is it true? It can be shown that by increasing the reinforcement ratio in a section, even in both the tension and compression faces, the displacement capacity of the wall is decreased. This implies that a weaker wall is as good as a more heavily reinforced or stronger wall. In fact, there may be less damage in the weaker or lightly reinforced wall.

An attempt has been made to investigate the performance of single structural walls under severe seismic loading. The main variable is the amount of longitudinal reinforcement in the section. To take

[1]Graduate Student, [2]Professor, Department of Civil Engineering, The University of British Columbia, Vancouver, BC, Canada, V6T 1Z4

into account the effects of axial load, different levels of compressive stress are considered, and to consider the effects of the periods of the structures, different height walls have been used. Also, sections with and without boundary elements have been considered.

STRUCTURAL WALLS USED IN THIS STUDY

Walls of 5, 10 and 15 stories, with 3.6 m story heights, are studied. The 5 and 10-story walls have a uniformly reinforced rectangular cross section, whereas the 15-story wall has boundary elements (Fig. 2). The axial load is assumed to give a compressive stress at the base level close to 0.05 f'_c and 0.1 f'_c, for the 5 and 10-story walls, respectively. The compressive stress on the boundary elements of the 15-story wall, assuming all vertical load is carried by the boundary elements, is 0.33 f'_c. The horizontal masses are assumed to be 2.5, 1.5 and 1.0 times of vertical masses for the 5, 10 and 15-story walls respectively. For rectangular sections, vertical reinforcement ratios ρ_v = 0.0025, 0.0050 and 0.0075 have been used, and for sections with boundary elements, the vertical reinforcement ratio in the wall panel ρ_v = 0.0025 and vertical reinforcement ratios in the boundary elements ρ_{col} = 0.01, 0.02 and 0.03 have been considered. The inelastic behavior is limited to flexural yielding, and Rayleigh damping of 5% in the first two modes is prescribed.

EQUIVALENT PLASTIC HINGE LENGTH

To make it possible to compare capacities and demands (capacities in terms of curvature and demands in terms of displacement) a definition for equivalent plastic hinge length, l_p, is necessary. For members with typical beam and column proportions, Paulay and Priestley (1992) have suggested that

$$l_p = 0.08\,H + 0.022\,f_y\,d_b \qquad [1]$$

where H is the distance between points of maximum and zero moment, d_b is the diameter of longitudinal reinforcement anchoring the member into a joint or footing, and f_y is the steel bar yield stress in MPa. They mention that the above equation gives an equivalent plastic hinge length close to 0.5 l_w, in which l_w is the length of the wall. Clearly, this will not apply to the stubby 5-story wall of this study.

Mattock (1967) in his discussion of Corley's paper (1966), suggests the following equation:

$$l_p = 0.5\,d + 0.05\,H \qquad [2]$$

where d is the effective depth of the section. He mentions that, for large values of d, his equation produces larger plastic hinge lengths than could be concluded from Corley's paper. Interpreting Corley's results in terms of l_p results in the following:

$$l_p = 0.5\,d + \frac{0.032\,H}{\sqrt{d}} \qquad [3]$$

In this equation all lengths are in meters. This equation, which gives a reasonable value for walls with different height to length ratios, has been used in this study. It is assumed that for the rectangular sections d = 0.8 l_w = 5.36m, and for the wall with boundary elements d = 6.35m. The equivalent - plastic lengths for the 5, 10 and 15-story structural walls become 2.93m, 3.18m and 3.86m respectively.

IDEALIZED MOMENT-CURVATURE RELATIONSHIPS

To define an elastic-perfectly plastic behavior for the sections, three parameters should be defined, effective flexural stiffness, yield or ultimate strength, and ultimate curvature. These parameters will be defined in this section, along with assumptions for material properties.

Material Properties and Modeling

Both longitudinal and transverse reinforcement are assumed to have f_y=410 MPa. To take into account overstrength of the reinforcement and its detrimental effect on ductility, a factor of 1.15 is applied to both f_y and f_u of the longitudinal steel (Paulay and Priestley 1992). Strain at rupture has been taken equal to 0.12. Figure 3a shows the stress-strain relationship for steel bars.

Concrete having a nominal compressive strength, f'_c, of 27.5 MPa is assumed where the maximum stress occurs at the strain of $\epsilon_o = 0.002$. The modified Kent-Park model is used to describe the stress-strain relation for both confined and unconfined concrete (Park et al 1982). To include tension stiffening in the effective flexural stiffness of sections, the average tensile concrete stress model suggested by Vecchio and Collins (1986) is used. These stress-strain relationships are shown in Fig 3b.

To define the available maximum concrete strain different equations have been introduced by Scott et al (1982) and by Kaar et al (1978) which lead to quite different results. For confinement as defined by ACI 318-89 these equations predict maximum usable concrete strains of 0.0208 and 0.0059 respectively. In this study available maximum concrete strains for unconfined and confined concrete are assumed to be 0.004 and 0.015 respectively.

Effective Flexural Stiffness of the Sections

It is assumed that for a wall with a moderate amount of vertical reinforcement, taken here to be $\rho_v = 0.5\%$ for the uniformly reinforced rectangular walls and $\rho_{col} = 2\%$ in the walls with boundary elements, $EI_{eff} = 0.5\ EI_g$ of the concrete section. This results in $T_1 = 0.46$s, 1.30s and 1.76s for 5, 10 and 15-story walls respectively. To alter EI_{eff} as the reinforcement ratio changes, the following procedure has been adopted.

Using the computer program BIAX (Wallace 1992) the moment curvature relations for sections with moderate reinforcement (and prescribed axial load), using the Park et al model (1982) for concrete in compression and Vecchio and Collins model (1986) for concrete in tension, are developed. The resulting effective moment-curvature for rectangular section with axial load equal to 6 MN, is shown by dashed curve in Fig. 4. The values of moment and curvature where first yield occurs, point A in Fig. 4, are used to define a secant stiffness, EI_{sec}. The ratio EI_{eff}/EI_{sec} derived for the moderately reinforced section and corresponding axial load, is then used to modify the EI_{sec} for walls with other reinforcement ratios to produce their EI_{eff}. The results for rectangular sections with P = 6 MN are shown in Fig. 5. The same procedure has been used for rectangular section with P = 3 MN and section with boundary elements with P = 9 MN. It should be mentioned that although the values of the flexural moments in the above calculations could be larger than real strength of the section, the calculated moment of inertia is a representative of the average cracked and uncracked moment of inertia over the length of the walls that have experienced yielding.

Ultimate Curvature of the Sections

The ultimate moment curvature, Φ'_u, is defined as the curvature when either (i) the strain of the concrete in compression reaches the maximum value; (ii) the reinforcing steel strain reaches its ultimate value (0.12); or (iii) the moment capacity reduces to less than 80% of the maximum capacity. Which ever of these three conditions occurs first defines the value of Φ'_u (Xiaoxuan and Moehle 1991). It has been observed that under cyclic loading ultimate curvature capacity is somewhat less than ultimate monotonic curvature capacity. Fajfar (1992) suggests the following equation to relate cyclic displacement ductility capacity, μ_Δ, to monotonic ductility capacity, μ'_Δ:

$$\mu_\Delta = \frac{\sqrt{1 + 4\,DM\,\beta\,\gamma^2\,\mu'_\Delta} - 1}{2\beta\gamma^2} \qquad [4]$$

In the above equation DM is the damage index, which in the case of safety limit design is equal to 1.0. Equation 4, which is based on low cycle fatigue using the Park-Ang damage model, implies the same relation between cyclic curvature capacity (Φ_u), and monotonic curvature capacity (Φ'_u). Fajfar suggests using average values of 1.0 and 0.15 for γ and β, respectively. Using these values, Eq. 4 reduces to

$$\mu_\Delta = 3.33\left(\sqrt{1 + 0.6\,\mu'_\Delta} - 1\right) \qquad [5]$$

Idiya and Bertero (1980) have tested different structural walls under monotonic and cyclic loading. It has been observed that for walls having monotonic displacement ductility capacities of $\mu'_\Delta = 5.0$ and 6.1, for the rectangular section and the section with boundary elements, respectively, the cyclic displacement ductility capacity has been decreased to μ_Δ=3.1 and 4.2. Equation 5 predicts μ_Δ=3.3 and 3.9, which are in good agreement with the test results.

To replace displacement ductility with curvature ductility, the following relation , suggested by Paulay and Uzumeri (1975), is used for the walls in this study:

$$\mu_\phi = 1 + \frac{\mu_\Delta - 1}{\dfrac{3l_p}{H'}\left(1 - \dfrac{l_p}{2H'}\right)} \qquad [6]$$

This equation was derived for a single degree of freedom mass on a cantilever column of height H'. For walls with an inverted triangular load distribution of height H, a similar expression might be used in terms of μ_Δ based on displacements at the top of the wall if H' is replaced by H and the coefficient 3.0 is replaced by 3.6.

Ultimate Bending Moment Strength of the Sections

Ultimate bending strength of the section is defined by the horizontal line that crosses the moment curvature curve at a point ⅔ Φ_p away from Φ_u as shown in Fig. 4 ($\Phi_p = \Phi_u - \Phi_y$). This has been applied to several walls with different amounts of reinforcement and appears to give reasonable estimates of yield strength to be used in an elastic-perfectly plastic model. Some of the idealized moment-curvature relationships are shown in Fig. 6. As it can be seen the higher the reinforcement ratio in the section, the less the ultimate curvature Φ_u.

DISPLACEMENT CAPACITIES AND DEMANDS

Assuming an inverted triangular distribution for the lateral load, the yield displacement at the top of the structural wall is $\Delta_y = \phi_y H^2/3.6$ if $\phi_y = M_y/EI$. Therefore the displacement capacity at the top of the structural wall is given by

$$\Delta_c^t = \frac{\Phi_y H^2}{3.6} + (\Phi_u - \Phi_y)\, l_p \left[H - \frac{l_p}{2} \right] \qquad [7]$$

To find displacement demand sixteen accelerograms all recorded on rock sites with PGA > 0.2g and PGV > 0.2 m/s^2 have been used in this study. Nonlinear time step analyses have been carried out with the elastic-plastic moment curvature relations using the computer program DRAIN-2DX. The ground motions were scaled to give displacement demands equal to displacement capacity for walls with a medium amount of reinforcement. The scaled records were then used to calculate the displacement demand for comparison to the displacement capacity for walls with lower and higher reinforcement ratios. Table 1 shows the results where the displacement demand is the average demand from the use of the sixteen records.

DISCUSSION

For 5, 10, and 15-story structural walls, the average scaling factors used on the records are 1.65, 2.15 and 8.17, respectively. If these walls with moderate amount of reinforcement were checked using NBCC (1990), for walls with 5, 10, and 15 stories the corresponding zonal velocity ratio, v, would be 0.40, 0.30, and 0.80, respectively. The average PGA and PGV of the sixteen records are 0.47g and 0.46 m/s. These values imply a very good performance of structural walls under severe earthquakes. The overall drift demands for 5, 10 and 15-story walls are 1/185, 1/144 and 1/43, respectively.

The results show that, for 10-story walls (with T_1 between 1.25s and 1.36s) and 15-story walls (with T_1 between 1.66s and 1.96s), displacement demands and displacement capacity are almost the same. In contrast, for 5-story walls (with T_1 between 0.43s and 0.49s) decreasing the amount of reinforcement in the section leads to poorer performance. Although lateral drifts for 5 and 10-story lightly reinforced walls are acceptable, for 15-story walls, reduction in reinforcement from ρ_{col}=.02 to ρ_{col}=.01 results in increasing overall drift from 1/43 to 1/37, both of which are large values.

CONCLUSION

Based on this limited investigation it is concluded that, except for short period structural walls, an increase in strength has little effect on the performance of the structure as measured by the curvature or displacement capacity. It should be noted that the stronger and the stiffer the wall, the smaller the lateral displacement. Thus, particularly in the case of sections with boundary elements, reduction in the amount of reinforcement might lead to unacceptable lateral displacement for the nonstructural elements. Perhaps it is time to consider displacement as a design criterium along with force.

This study has shown the very good performance of structural walls against severe earthquake, as noted by other authors.

REFERENCES

Corley, W.G. 1966. Rotational Capacity of Reinforced Beams. Journal of the Structural Division, ASCE, Vol. 92, No. ST10, 121-146.

Fajfar, P. 1992. Equivalent Ductility Factors, Taking into Account Low-cycle Fatigue. Earthquake Engineering and Structural Dynamics, Vol. 21, 837-848.

Iliya, R., Bertero, V.V. 1980. Effects of Amount and Arrangement of Wall-Panel Reinforcement on Hysteretic Behavior of Reinforced Concrete Walls. UCB/EERC-80/04, Department of Civil Engineering, University of California, Berkeley, CA.

Kaar, P.H., Fiorato, A.E., Carpenter, J.E., Corley, W.G. 1978. Limiting Strains of Concrete Confined by Rectangular Hoops. Research and Development Bulletin RD053, Portland Cement Association, Skokie, IL.

Mattock, A.H. 1967. Discussion of Rotational Capacity of Reinforced Concrete Beams, by W.G. Corley. Journal of the Structural Division, ASCE, Vol. 93, ST2, 519-522.

NBCC. 1990. National Building Code of Canada. Association committee on the National Building Code, National Research council of Canada, Ottawa, Ont.

Park, R., Priestley, M.J.N., Gill, W.D. 1982. Ductility of Square Confined Concrete Columns. Journal of the Structural Division, ASCE, Vol. 108, No. ST4, 929-950.

Paulay, T., Uzumeri, S.M. 1975. A Critical Review of the Seismic Design Provisions for Ductile Shear Walls of the Canadian Code and Commentary. Canadian Journal of Civil Engineering, Vol. 2, No. 4, 592-601.

Paulay, T., Priestley, M.J.N. 1992. Seismic Design of Concrete and Masonry Structures. John Wiley and Sons, New York, NY.

Scott, B.D., Park, R., Priestley, M.J.N. 1982. Stress-Strain Behavior of Concrete Confined by Overlapping Hoops at Low and High Strain Rates. ACI Journal, Vol. 79, No. 1, 13-27.

Vecchio, F.J., Collins, M.P. 1986. The Modified Compression-Field Theory for Reinforced Concrete Elements Subjected to shear. ACI Journal, Vol. 83, No. 2, 219-231.

Wallace, J.W. 1992. BIAX: A Computer Program for the Analysis of Reinforced Concrete and Reinforced Masonry Sections. CU/CEE-92/4, Department of Civil Engineering, Clarkson University, Potsdam, NY.

Xiaoxuan, Q., Moehle, J.P. 1991. Displacement Design Approach for Reinforced Concrete Structures Subjected to Earthquakes. UCB/EERC-91/02, Department of Civil Engineering, University of California, Berkeley, CA.

TABLE 1. Displacement demands and capacities (in meters)

No. of Stories of Structural Wall	Section Type	ρ_v =.0025 (Rectangular Section) ρ_{col}=.01 (Barbell Section)		ρ_v =.0050(Rectan.) ρ_{col}=.02 (Barbell)	ρ_v =.0075(Rectangular Section) ρ_{col}=.03 (Barbell Section)	
		Light Reinforcement		Moderate Reinforcement	Heavy Reinforcement	
		Capacity	Demand	Capacity = Demand	Capacity	Demand
5	Rectan.	0.109	0.126	0.0975	0.0905	0.0814
10	Rectan.	0.266	0.267	0.250	0.243	0.248
15	Barbell	1.510	1.460	1.270	1.140	1.180

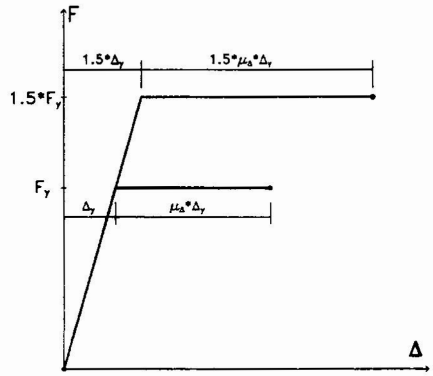

FIGURE 1. Force-displacement relationships for constant displacement ductility

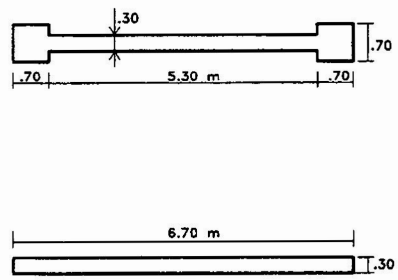

FIGURE 2. Rectangular section and section with boundary elements (barbell section)

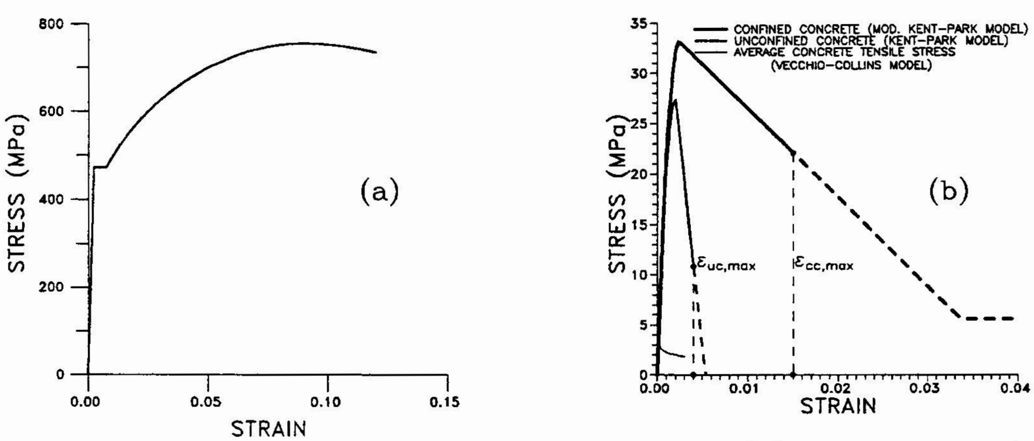

FIGURE 3. Stress-strain relationships for steel (a) and concrete (b)

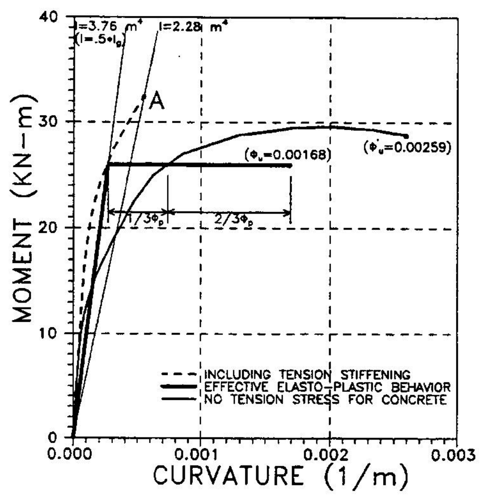

FIGURE 4. Moment-curvature relationships for rectangular section with axial load equal to 6 MN (f_c=0.11 f'_c) and ρ_v=0.0050 (moderately reinforced)

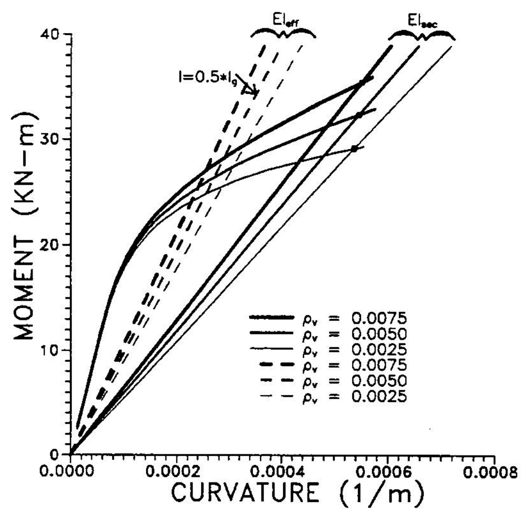

FIGURE 5. Moment-curvature relationships for rectangular sections with axial load equal to 6 MN and corresponding effective and secant stiffness

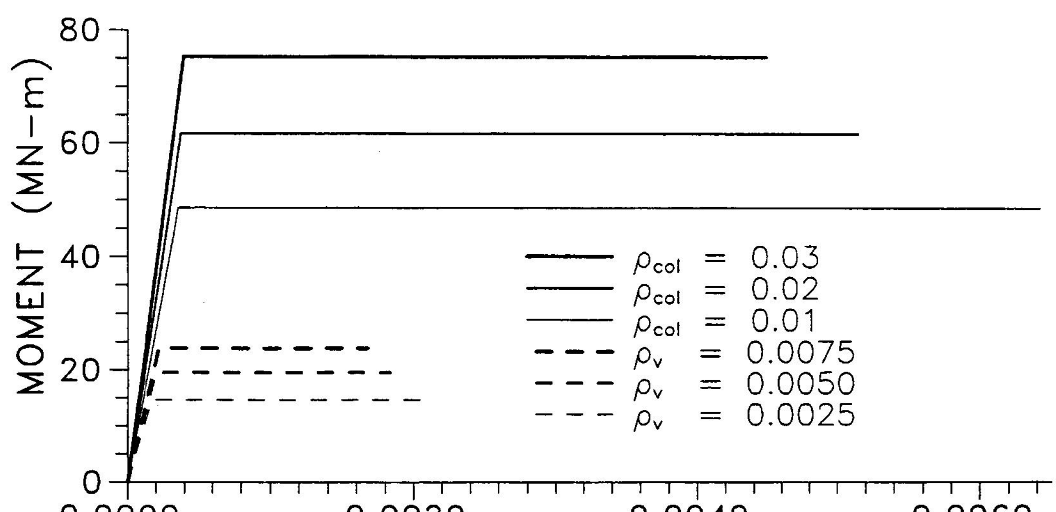

FIGURE 6. Idealized moment-curvature relationships for section with boundary elements (barbell section) with P=9 MN (solid lines) and rectangular section with P=3 MN (dased lines)