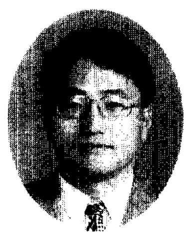
전단벽의 최소 층변위 및 에너지 소산성능

The Limiting Drift and Energy Dissipation Ratio for Shear Walls Based on Structural Testing



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요 약

현재 미국에서는 강한 지진지역에서의 골조구조에 대한 새로운 실험규정이 만들어지고 있으며, 이의 목적은 비교적 신뢰성이 높은 실험결과를 얻고 이들 실험결과를 다른 연구자들이 서로 이용 가능하도록 하는 것이다. 이 실험규정에서는 실험방법 뿐만아니라, 실험후의 분석방법 특히, 실험체가 최소한 보유하여야 할 층변위각, 에너지 소산성능, 강성, 강도 등이 규정되어 있다. 이러한 지침이 설정됨으로 인하여, 여태까지 주관적으로 평가된 실험결과의 분석들이 비교적 객관적으로 평가될 수 있게 될 것으로 보여진다.

전단벽 구조 역시 지진저항에 매우 효과적인 시스템으로서, 이러한 실험지침이 필요하다. 그러나, 전단벽 구조의 주 부재인 전단벽은 횡력에 의해서 발생하는 구조물의 횡변위를 억제시키고, 강성과 강도를 증가시키는 역할을 하기 때문에 그 거동 특성이 골조구조와는 다소 다르다.

본 연구에서는 이러한 전단벽의 층변위와 에너지 소산성능에 대하여 연구를 하고 구조실험시 요구되는 적정 값들을 제시하고자 하였다. 구조실험시 (반복하중실험), 높은 지진지역의 전단벽 구조가 보유해야할 최소 변형능력 (횡변위)을 구하기 위해, 기존 연구자들에 의해 실험된 일련의 실험자료들을 분석할 뿐만아니라, 전단벽을 켄틸레버로 이상화 하여 층변위를 형상비, 변위 연성비의 관계로 나타내고, 현재 각 국가의 내진설계 규정에서 정하고 있는 건물의 층변위각을 고려하여 전단벽의 최소 층변위를 제시하였다. 또한, 미국의 NEHRP 규준에서 규정하고 있는 소산에너지와 감쇠의 관계를 이용하고, 변위 연성비를 도입하여 구조실험시 요구되는 전단벽의 최소 소산에너지값을 제시하였다.

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1. Introduction

Structural walls are very effective elements for resisting earthquake loads. Extensive experimental and analytical research have been conducted aimed at determining the factors controlling the behavior of structural walls and verifying procedures as to how to design them effectively. The need to specify guidelines or criteria for the acceptance of walls based on structural testing has been raised by some researchers[1]. Such guidelines would allow more direct comparisons between test results from different laboratories and provide more reliable information from which to develop design rules. The importance of such guidelines is apparent from the following finding on strucutral wall testing that was reached in a 1997 NSF sponsored workshop[2]: It is difficult to make valid comparisons of available ductility values reported by different researchers because they are often based on different response parameters or on yielding values determined using definitions that are different or unexplained or both.

Recently, a Provisional Standard for acceptance criteria[3], based on the experimental evidence and analysis, has been developed that establish dependable and predictable strength, drift capacity and relative energy dissipation requirements for strong column/weak beam moment frames in the regions of high seismic risk. This Provisional Standard envisages that precast and/or prestressed concrete moment frames satisfying its requirements also satisfy the requirements of 21.2.1.5 of ACI 318-95 and have strength and toughness equal to or exceeding those provided by conventional monolithic reinforced concrete frames satisfying the requirements of 21.2 through 21.6 of ACI 318-95. The Provisional Standard specifies, in addition to acceptance criteria, procedures for the design of the modules required for the validation testing of a generic type of frame, the number and type of test modules required, test methods and test report content. A specific limiting drift ratio for the test modules and a corresponding limiting relative energy dissipation ratio are specified.

In a manner similar to that for frames, it is desirable that acceptance criteria, that meet the requirements of 21.2.1.5 of ACI 318-95 also be developed for structural walls. The objective of this paper is to initiate the development of such criteria by proposing limiting drift and energy dissipation ratios for structural walls based on examination and analysis of available test data.

2. General Behavior of Shear Walls

Shear walls are typically stiff and, therefore, tend to prevent the large deformations that can be a problem for attached nonstructural components. However, under significant lateral seismic excitation, a shear wall can fail by a variety of mechanisms, resulting in large lateral displacements and loss of stiffness and strength. The behavior of shear walls depend on the various section properties such as aspect ratios, section shapes, axial forces, reinforcement details, etc. The displacement is less sensitive to the shapes and reinforcement details of boundary elements than the strength of shear walls. A research result[4] shows that the displacement of frame structure with walls is very similiar to that of shear walls

3. Limiting Drifts

The limiting drifts have been classfied for two states, the serviceability level and ultimate safety level, by Bertero[1]. At the service level, the present seismic codes put maximum limit in the range of 0.06 - 0.6%[1]. The limiting drift at the service level is not the concern in this paper. The foregong values for each code are ultimate limits. Bertero[1] reports that the usual variations in interstory drifts for structures designed to present seismic codes are in the range of 1 to 3 percent and vary with the type of structure and its function.

3.1 Required limiting drifts

The process adopted in the provisional standard for acceptance criteria for frame structures[3] is used here to suggest the required limiting drift for the characteristics of the shear walls which codified in UBC 1994 and NEHRP provision 1994 as a structural system.

The story drift limits specified in UBC 1628.8 are

$T < 0.7$ sec, $\delta = (0.04/R_w)$ H or 0.005H

$T > 0.7$ sec, $\delta = (0.03/R_w)$ H or 0.004H

Where, T = fundmental period, R_w = numerical coefficient, 12 for special moment-resisting frames of moment resisting frame system (MRFS), 6 for shear walls of bearing wall system (BWS), 8 for shear walls of building frame system (BFS), H = height of structure.

The deformation compatibility of UBC 1631.24 is $3(R_w/8)$ x calculated drift. However, Uang[5] has suggested that the upper bounds for expected drifts is not (3/8) R_w but 1.0 R_w for frame structures. Futher, Veletos[6] has reported that the drift of an inelastic structure is about that of an elastic structure with same initial period. Table 1 shows corresponding limiting drifts for three different structural forms specified in UBC 1994. It is difficult for a conventional moment frame, designed to UBC 1994, to achieve a drift of 4 percent without failure. Further, at that value, nonstructural damage is very high. Therefore a value of 3.5 percent was suggested as the limiting drift ratio for SMRF[3]. Even though values of 3 percent for shear walls of BWS and 4 percent for shear walls of BFS are listed in Table 1, those values are unachieveable for shear walls with low aspect ratios. It is not easy to specify an acceptable limiting drift ratio for shear walls, because drifts are very senstive to the sectional characteristics of the wall.

3.2 Limiting drift based on the analysis of test results

Duffey[7] reviewed experimental data for squat shear walls with aspect ratios between 0.24 and 1.07 and, based on statistical analyses, suggested the limiting drifts shown in Table 2 and Fig.1.

Table 2 Drift variation with post-peak load[7]

Fraction of Peak Load (%)	100	90	80	70	60	50
Drift (%)	0.72	1.00	1.24	1.48	1.64	1.84

Limiting drifts are shown as the function of the fraction of the post-peak ultimate load. For either frames or walls the limiting strength, for stability reasons, is customarily taken as the strength when the load has decreased to 80 or

Table 1 Drift angle according to UBC 1994

System	R_w	δ	3/8 R_w δ(%)	1.0 R_w δ(%)	Suggested Value*
MRFS	12	0.00333H	1.5	4.0	3.5
Shear Wall of BWS	6	0.005H	1.125	3.0	To be suggested
Shear Walls of BFS	8	0.005H	1.5	4.0	To be suggested

* Suggested value for structural form for acceptance testing

to 85 percent of the peak load[8, 3]. Table 2 and Fig.1 show that limiting drifts for that condition are between 1.12 and 1.24 percent.

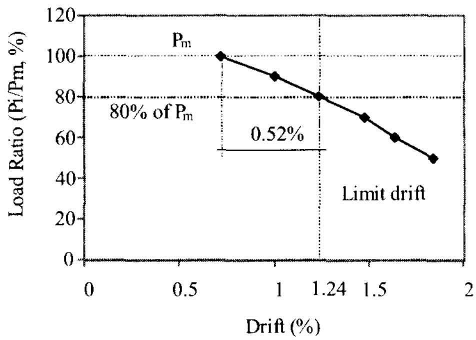


Fig.1 Drift variation with post-peak load[7]

Table 3 List of data

Researcher	No.	Aspect Ratio	Loading*	Ref.
Wiradinata	4	0.25, 2.5	C	7
Saatcioglu	8	0.25, 0.5	C	7
Shiga	14	0.68	C	7
Endo	20	1.0	C	7
Paulay	3	0.5	C	7
Alexander	2	0.75	C	7
Barda	11	0.24, 0.51, 1.07	C, M	7
Benjamin	19	0.32, 0.50, 0.58	M	7
Williams	17	0.32, 0.50, 0.58, 0.69, 0.71, 1.25	M	7
Cervenka	3	1.0	M	7
Corley	19	2.69, 3.53	C, M	7
Elnashai	16	2.0	C	7
Flonato	4	2.40, 2.69	C	7
Lefas	13	1.0, 2.0	M	7
Yamada	7	0.44	M	7
Maier	11	1.22	C, M++	7
Hiraishi	4	1.7	C	9
Bertero	2	1.28	C, M	10
Bertero	4	1.28	C, M	11
Morgan	1	2.78	C	12
Total	178	0.25~3.53		

* Loading pattern
(C : Cyclic loading, M : Monotonic loading, M++ : Some reloading, not cycle)

Duffey[7] also analyzed specific test results to determine limiting drift values. His data are again analyzed here with data where drifts were controlled artificially to 1.0 percent excluded and with other test data appended. Table 3 lists the data analyzed in this paper. Results for 178 walls with aspect ratios between 0.25 and 3.53 are used.

Fig. 2 shows the statistical analysis result.

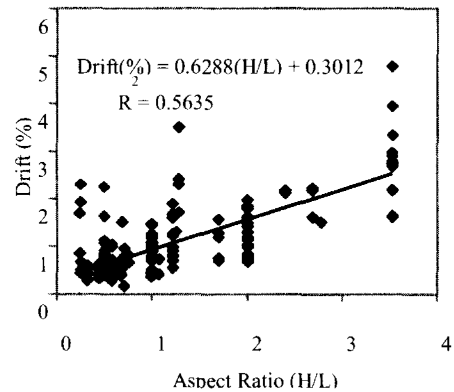


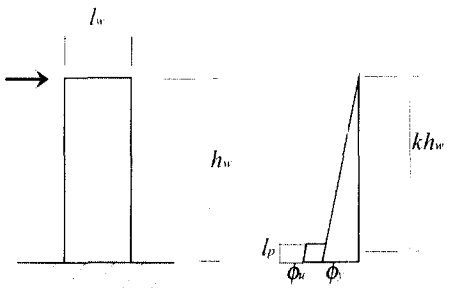
Fig. 2 Drift-aspect ratio relation

The limiting drift varies linearly with the aspect ratio. The drift increment between peak load and the failure load, equal to 80 percent of peak load, should also be considered, because drifts in Table 3 are those at peak load. That effect can be approximated by using the result of Fig. 1 that post-peak load drifts increase almost linearly with decreasing post-peak loads. Eq. (1) is the resultant expression relating limiting drifts and aspect ratio for shear walls.

$$Drift\,(\%) = 0.6288\left(\frac{H}{L}\right) + 0.8212 \qquad (1)$$

3.3 Maximum drift by a simple method

A procedure for relating displacements to local inelastic demands can be based on the model of Fig. 3. Elastic curvatures vary over the wall height in proportion to wall moment. For inelastic response, the maximum elastic curvature is equal to the yield curvature ϕ_y, which is the curvature at first yield of the wall boundary reinforcement. Inelastic curvatures up to the maximim curvature accumulate at the base of the wall along a height l_p resulting in a plastic hinge rotation θ_p.



Cantilever shear wall   Curvature distribution

Fig. 3 Curvature of cantilever shear wall

The displacement at the top of the wall can be approximated by Eq. (2).

$$\delta_u = \delta_y + \theta_p h_w \tag{2}$$

$$= \frac{1}{3}(\phi_y h_w^2) + (\phi_u - \phi_y) l_p k h_w \tag{3}$$

$$\mu_\delta = \frac{\delta_u}{\delta_y} = 1 + \frac{(\phi_u - \phi_y)}{\frac{1}{3}(\phi_y h_w^2)} l_p k h_w \tag{4}$$

$$\frac{\phi_u}{\phi_y} = \frac{l_p k h_w}{l_p k h_w} + \frac{(\mu_\delta - 1) h_w^2}{3 l_p k h_w} = \mu_\phi \tag{5}$$

$$\phi_u = \frac{(u_\delta - 1) h_w^2}{3 l_p k h_w} \phi_y + \phi_y$$

The length of plastic hinge and yield curvature can be expressed as the function of the wall length.

$$l_p = \alpha l_w, \quad \phi_y = \frac{\beta}{l_w}$$

From Eq.(3), the curvature ductility is

$$\delta_u = \frac{1}{3}\left(\frac{\beta}{l_w} h_w^2\right) + \left(\phi_u - \frac{\beta}{l_w}\right) \alpha l_w k h_w \tag{6}$$

$$\frac{\delta_u}{h_w} = \frac{\beta}{3} \frac{h_w}{l_w} + \left(\phi_u - \frac{\beta}{l_w}\right) \alpha l_w k \tag{7}$$

$$\frac{\delta_\mu}{h_w} = \frac{\beta u_\delta}{3} \left(\frac{h_w}{l_w}\right) \tag{8}$$

Where, δ_u = displacement at ultimate load, δ_y = yield displacement, θ_p = plastic deflection angle, h_w = height of structure, ϕ_y = yield curvature, ϕ_u = curvature at ultimate load, l_p = plastic hinge length, μ_δ = displacement ductility at ultimate load, μ_ϕ = curvature ductility at ultimate load, β = a value expresssing the relation of yield curvature ϕ_y and depth of section

Eq. (8) involves both ductility and aspect ratio. However, the drift, δ_u/h_w of Eq. (8) is the value at the ultimate load. That value needs to include the additional post-peak drift. From Fig. 1 it can be seen that drift is 0.52 percent so that Eq. (8) becomes :

$$\frac{\delta_m}{h_w} = \frac{\beta \mu_\delta}{3} \left(\frac{h_w}{l_w}\right) + \frac{1}{192} \tag{9}$$

In Eq.(9), the value of factor β increases with increasing steel ratio and axial load, and is not easy to be approximated. For design purpose, a value of 0.0025 is suggested[13]. Shown in Fig. 4 is the resultant variation in drift with the aspect ratio and ductility demand calculated from Eq. (9).

The commentary NZS 4203[14] defines an approximate criterion for the limiting dispalacement of shear wall structures such that the building as a whole can sustain four cycles of loading with a ductility ratio μ of 8 without the base shear decreasing by more than 20 percent. Even though the structural ductility factor is required to be modified according to the period, it is reasonable to a use maximum ductility factor 8 for assessing a limiting drift based on structural testing.

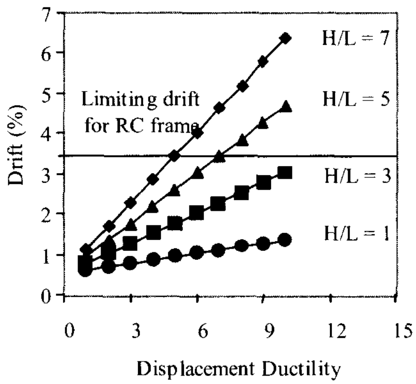


Fig. 4 Drift-displacement ductility curve

Then Eq. (9) can be changed to Eq. (10) with the value β of 0.0025.

$$\frac{\delta_m}{h_w}(\%) = 0.67\left(\frac{h_w}{l_w}\right) + 0.52 \tag{10}$$

Where, δ_m = maximum displacement at the 0.80 P_m.

3.4 Limiting drift ratios for shear walls

Two equations for the limiting drifts of shear walls have been proposed by analyzing the available test data and using a simple curvature theory. Fig. 5 shows that the curve of Eq. (1) agrees wtih that of Eq. (10). An expression for the limiting drift ratio of a shear wall can be derived by taking 3.0 percent drift as the upper bound. That drift is also the value for 1.0R for shear walls for BWS in Table 1. Also the least drift should be more than 1 percent to satisfy the limiting building drifts for each design code.

Thus, an appropriate limiting drift expression for shear walls is,

$$1 \le \frac{h_m}{\delta_m}(\%) = 0.67\left(\frac{h_w}{l_w}\right) + 0.52 \le 3 \tag{11}$$

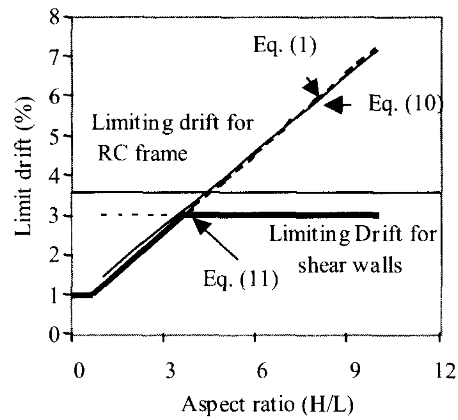


Fig. 5 Limiting drift ratio for shear walls

4. Relative energy dissipation ratio for shear walls

In the acceptance criteria[3], the limiting energy dissipation ratio for RC frame structure is set at 12 percent for the third complete cycle at or exceeding the maximum drift of 3.5 percent. This limit is to prevent the low cycle fatigue effects caused by the inadequate damping and obviate possible dieplacement increases with cycling. Energy dissipation is best expressed as an equivalent viscous damping. In this paper, a procedure based on the relation between energy dissipation ratio and damping ratio is adopted to suggest a limiting energy dissipation ratio for shear walls based on structural testing.

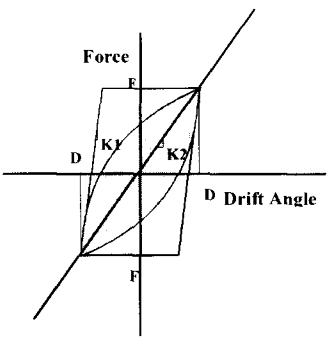


Fig. 6 Relative energy dissipation

Fig. 6 shows the hysteretic loop of the acceptance criteria[3]. The area of the parallelogram in Fig. 6 is :

$$(2D-2x_1)\times 2F=4(D-x_1)F \tag{12}$$

$$Here,\quad K_1=F/D,\quad x_1= D\frac{K_2}{K_1}$$

Eq. (12) may be written as :

$$4\,(D-x_1D)K_1=4\left(D-D\frac{K_2}{K_1}\right)DK_1 \tag{13}$$

Energy dissipation ratio is

$$\frac{W}{\Delta W}=\frac{A_h}{4K_1D^2(1-\frac{K_2}{K_1})} \tag{14}$$

$$\beta_I=\frac{A_h}{2\pi\,K_1D^2}$$ From the 94' NEHRP Provision

Where, A_h = total area at a cycle and β_I = effective damping.

Eq. (13) can be rewritten as the relationship between energy dissipation ratio and damping ratio.

$$\frac{A_h}{2\pi K_1D^2}\times\frac{\pi}{2\left(1-\frac{K_2}{K_1}\right)}=E_R \tag{15}$$

$$\beta_I\times\frac{\pi}{2\left(1-\frac{K_2}{K_1}\right)}=E_R \tag{16}$$

E_R = energy dissipation ratio

From Fig. 6,

$$\frac{K_2}{K_1}=\frac{x_1}{D}=\frac{\delta_y}{\delta_m}=\frac{1}{\mu_\delta} \tag{17}$$

The values of μ_δ are 10 and 8 for concrete frames and shear walls according to the NZS 4203[14] so that the energy dissipation ratios are

$$E_R=1.744\beta_I\ and\ 1.794\beta_I$$

Here, these values are coefficients to represent the relationship between damping and energy dissipation ratio at maximum drift angle. The values of equivalent viscous damping ranging from 5 for steel frames to 7 or 8 percent for shearwalls[15, 16] have been reported. Also Dowrick[17] has reported 5 and 10 percent as the damping ratios of steel frames, concrete frames and shear walls.

The minimum energy dissipation ratio specified in acceptance criteria[3], are reviewed in here. The damping ratio of RC frame structure can be estimated directly from the 12 percent energy dissipation ratio.

$$\beta_I=\frac{12(\%)}{1.744}=6.88(\%)$$

This value is between 5 and 7 and consistent with the damping ratios for concrete frames, have been reported by Dowrick[17]. If damping ratio of shear walls can be assumed 8.5 percent which is average of values of 7, 8, and 10 percent[15, 16, 17], the limiting energy dissipation ratio for shear walls is

$$E_R=1.794\times(8.5\%)=15.3\% \tag{18}$$

5. Conclusion

Appropriate limiting drift and energy dissipation ratios for shear walls for post-peak loads values equal to 80 percent of the ultimate load were studied. To derive a suitable limiting drift ratio for shear walls, available test data were analyzed and results compared to drift values calculated using a simple cantilever method. The result is a formula for drift that is a function of aspect ratio. A limiting energy dissipation ratio for shear walls was derived by using accepted damping ratio for structures. The limiting energy dissipation ratio for shear walls was 15.3 percent at a displacement ductility ratio of 8 and a damping ratio of 8.5 percent.

Acknowledgments

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ABSTRACT

Recently, new experimental criteria for reinforced concrete frame structures in high seismic regions have been reported in United States[3]. The objective of the criteria is to get more reliable test data which are valid to compare with other test data done by different researchers. The criteria precribe test method of specimens, analysis method of test data, and limiting values needed to specimens like drift angle, energy dissipation ratio, stiffness, and strength. These criteria might be useful to get objective conclusion. Shear wall structures, which belong to one of earthquake resisting systems, also need this kind of criteria. But, the general response of shear wall structures is a little bit different from that of frame structures since shear wall restrains the horizontal displacement caused by horizontal force and increases the stiffness and strength.

The objective of this paper is to propose a criterion for limiting drift and energy dissipation ratio of shear walls based on structural testing. These are the most important values for presenting the capacity of shear walls. Limiting drift and energy dissipation ratios were examined for tests on shear walls having ductile type failures. Test data were analyzed and compared to the results for a suggested acceptance criteria that involve a limiting drift that is a function of aspect ratio and a limiting energy dissipation ratio that is a function of displacement ductility and damping.

Keywords: Criteria, Limiting drift, Energy dissipation ratio, Shear walls

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