TESTS OF STRUCTURAL WALLS UNDER REVERSING LOADS

by

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ABSTRACT

Tests of 14 isolated structural walls subjected to reversing in-plane horizontal loads are described. Controlled variables included shape of the wall cross section, amount of flexural and shear reinforcement, confinement reinforcement in the boundary elements, axial load, and concrete strength. In addition, one wall was repaired and retested.

The tests were made to evaluate hysteretic response, strength and deformation capacity of structural walls used for lateral bracing in earthquake-resistant buildings. This paper includes discussions of the observed response of the specimens, the observed and calculated strengths, and the observed deformation capacities.

INTRODUCTION

Structural walls are frequently used as stiffening elements for wind and earthquake resistance in tall buildings. For severe earthquakes, a structure cannot reasonably be designed to remain elastic. Rather, inelastic response must be considered in design.

Tests described in this paper provide information on the inelastic hysteretic response of isolated structural walls. In particular, the importance of selected controlled variables on the strength and deformation capacity of the test specimens was investigated.

EXPERIMENTAL PROGRAM

Test specimens represented approximately 1/3-scale models of full-size walls, although no particular prototype was modeled. Controlled variables included shape of the wall cross section, amount of flexural and shear reinforcement, confinement reinforcement in the boundary elements, axial load, concrete strength and load history. Table 1 gives a summary of the test program.

Test Specimens

Dimensions of the test specimens are shown in Fig. 1. Rectangular, barbell and flanged cross sections were tested. Nominal cross-sectional dimensions of these sections are shown in Fig. 2. A cross section showing locations of the types of reinforcement used is shown in Fig. 3.

The design moment for each wall was calculated following the 1971 ACI Building Code (American Concrete Institute, 1971). Design yield stress of the flexural reinforcement was 60 ksi (414 MPa). In proportioning the steel, strain hardening was neglected. Design concrete strength was 3000 psi (20.7 MPa) for Specimen B6 and 6000 psi (41.4 MPa) for all other specimens.

Several criteria were used to select horizontal shear reinforcement. Minimum requirements of the 1971 ACI Building Code (American Concrete Institute, 1971) governed for the first five specimens in Table 1. For B2, B5 and F1 horizontal reinforcement was designed using a shear force corresponding to the calculated design moment. Shear reinforcement was provided in accordance with the 1971 ACI Building Code (American Concrete Institute, 1971). Specimens B6, B7 and F2 were provided with the same amount of shear reinforcement as B2 and B5.

To determine the influence of shear reinforcement, a different design procedure was used for Specimen B8. A shear force corresponding to the calculated maximum moment capacity of the wall including strain hardening of the vertical reinforcement was used. The horizontal shear reinforcement was selected to carry this entire shear force at a design yield stress of 60 ksi (414 MPa).

Transverse reinforcement around vertical reinforcement in the boundary elements was designed either as ordinary column ties (unconfined) or as special confinement reinforcement (confined). For rectangular sections, the "boundary element" was taken to extend 7.5 in. (190 mm) from each end of the wall.

Specimens R1, B1, B2, and F1 had ordinary ties as required by Section 7.12 of the 1971 ACI Building Code (American Concrete Institute, 1971).

All other specimens had rectangular hoop and supplementary cross-tie reinforcement in accordance with Appendix A of the 1971 ACI Building Code (American Concrete Institute, 1971). This design resulted in a hoop spacing of 1.33 in. (34 mm).

Confinement was used only over the first 6 ft (1.83 m) above the base of the wall. Ordinary column ties were used over the remaining height. Specimen F2 had a special "boundary element" within the intersection of the web and the flange at each end of the wall. The confined zone extended into the web 12 in. (305 mm) from the end of the wall and into the flange 6 in. (152 mm) on either side of the centerline of the web.

Specimen B5R was a retest of B5. Following the test of B5, damaged web concrete was removed up to a height of about 9 ft (2.74 mm). New web concrete was cast in three lifts. The columns were repaired with a surface coating of neat cement paste.

Test Procedure

The test setup for the walls is shown in Fig. 4. Each specimen was loaded as a vertical cantilever with forces applied through the top slab. The shear span was 2.4 times the horizontal length of the wall. For all specimens, except B4, reversing horizontal loads were applied in a series of increasing increments. Each increment consisted of three completely reversed cycles. About three increments of force were applied prior to initial yielding. Subsequent to initial yielding, loading was controlled by deflections in 1.0 in. (25 mm) increments. Specimen B4 was subjected to a monotonically increasing load.

Constant axial compressive loads were maintained on Specimens B6, B7, B8, and F2. These loads were applied such that the resultant axial force remained vertical throughout the horizontal load cycles.

A more detailed description of the experimental program is given elsewhere (Oesterle, et.al., 1976 and Oesterle, et.al., 1977).

OBSERVED BEHAVIOR

Vertical flexural reinforcement in the specimens was selected so that behavior in two ranges of response could be observed. These were distinguished by the magnitude of the maximum nominal shear stress applied to the specimen.

Walls Subjected to Low Nominal Shear Stress

Specimens R1, R2, B1, and B3 were subjected to maximum nominal shear stresses less than $3.1\sqrt{f'_c}$ psi ($0.26\sqrt{f'_c}$ MPa). Cracks in these walls started as horizontal flexural cracks in the boundary element. Closely spaced confinement hoops caused the cracks to be finely distributed. The horizontal cracks progressed into coarsely distributed inclined cracks in the web. Intersecting cracks from opposite loading directions segmented the lower wall region into several horizontal layers as illustrated in Fig. 5.

Because of the horizontal cracks, little truss action developed to transmit shear. After yield, shear was transferred primarily by interface friction, bearing of particles on opposite sides of the crack, and dowel action. This mode of shear transfer was adequate to maintain the integrity of the walls.

Capacity of the walls with low nominal shear stress was limited by alternate tensile yielding and compressive buckling of the main vertical reinforcement. The buckling was accompanied by loss of concrete not contained by the reinforcement. Buckling of vertical steel was followed, after several cycles, by bar fracture. One or two bars fractured at a time with a corresponding decrease in load. Confinement hoops around the main vertical reinforcement delayed bar buckling and contained the core concrete.

Figure 5 shows the hysteretic response of Specimen B3 in terms of the applied load versus top deflection.

In Specimen R2, a rectangular wall, large out-of-plane displacements of the compression zone were observed as the specimen was cycled in the inelastic range. These displacements were caused by alternate tensile yielding of the main vertical reinforcement. As the wall was loaded beyond yield in one direction and then unloaded, permanent deformations remained in the tension steel. When the load was reversed, compression was carried primarily by the vertical steel. Thus the stiffness of the compression zone was reduced considerably from that of the elastic uncracked section, and stability problems were encountered.

Walls Subjected to High Nominal Shear Stress

Nine specimens were subjected to maximum nominal shear stresses greater than $7.0\sqrt{f'_c}$ psi ($0.58\sqrt{f'_c}$ MPa). Flexural cracks, first observed in the boundary elements, propagated into inclined web cracks. Reversed loading resulted in a system of inclined cracks that crisscrossed the web forming relatively symmetrical compressive strut systems for each direction of loading. Each compressive strut was segmented into parallelogram shaped pieces of concrete as can be seen in Fig. 6.

In the walls subjected to high nominal shear stress, the inclined struts formed the primary shear resisting mechanism through truss action. As the specimens were repeatedly cycled in the inelastic range, the concrete segments in the struts were abraded. Loss of concrete by abrasion and by surface spalling increased shear deformations and reduced the compressive strength of the struts. There were also visual indications that local inelastic buckling of wall reinforcement contributed to deterioration of the struts. The capacity of the walls was finally limited by web crushing.

Confinement reinforcement in the boundary elements increased the shear capacity and stiffness of the boundary elements.

An example of the hysteretic response of a wall subjected to high nominal shear stress, Specimen B5, is shown in Fig. 6.

OBSERVED VERSUS CALCULATED STRENGTHS

Full yield and maximum strengths observed during the tests are summarized in Table 2. Values are given for the measured load and for the corresponding nominal shear stress. Yield loads reported are those applied when all flexural reinforcement in the boundary element reached yield.

Design Strengths

Design strengths listed in Table 2 ~~were~~ calculated following the 1971 ACI Building Code (American Concrete Institute, 1971) considering the capacity reduction factor, $\varphi = 1.0$. In Fig. 7 observed strengths are plotted versus the design strengths. Observed loads for all specimens exceeded the ACI design strengths.

Because of the assumptions involved in the ACI design procedure, the design flexural strength corresponds more closely to yield than to maximum. This is primarily because strain hardening of the reinforcement is neglected. For seismic design, the difference between yield and maximum strength is important. Response of the structure to severe ground motions may induce shear forces corresponding to moments, that are produced when strain hardening occurs.

Calculated Strengths

Calculated maximum loads listed in Table 2 are based on monotonic flexural strengths considering strain compatibility using measured material properties including strain hardening of the reinforcement.

Specimens subjected to low maximum nominal shear stresses had strengths ranging from 84% to 91% of the calculated maximum flexural strength. By comparison, B4, loaded monotonically had a maximum observed strength equal to 101% of that calculated. The companion to B4, Specimen B3, was loaded cyclically. It reached 84% of its calculated monotonic strength and 80% of the observed strength of B4. Differences between observed and calculated loads for specimens failing in flexure are attributed to the effects of load reversals. In particular, repeated cycles of compressive buckling and then straightening of vertical reinforcement in the inelastic range of behavior reduced the strength of the steel in relationship to its monotonic tensile strength.

Specimens subjected to high maximum nominal shear stresses had strengths limited by web crushing. Addition of axial compressive load increased the maximum strength as can be seen by comparing B5 and B7 in Table 2. However, addition of horizontal shear reinforcement did not significantly affect web crushing strength. This is evident from a comparison of B7 and B8.

DEFORMATION CAPACITY

In addition to strength, the deformation capacity of the walls is particularly important for seismic design. Two measures of inelastic deformation capacity used to evaluate the performance of the walls are deflection ductility and energy dissipation.

Ductility

Figure 8 shows the maximum deflection ductility ratio versus the maximum nominal shear stress observed for each specimen. The cyclic ductility ratio is based on the measured deflection at the top of each specimen. It is defined in the inset of Fig. 8. The maximum ratio is that at the last stable load increment. This was defined as an increment in which at least 80% of the previous maximum load was sustained in all three cycles.

It is apparent from Fig. 8 that the ductility of the walls decreased with increasing nominal shear stress. The strength of walls with shears greater than $7.2\sqrt{f'_c}$ psi ($0.60\sqrt{f'_c}$ MPa) was limited by web crushing. Comparison of B5 and B7 in Fig. 8 indicates that addition of axial compressive load increased both ductility and strength of the barbell type wall that failed by web crushing. Addition of horizontal shear reinforcement in B8 did not significantly affect its ductility in comparison to B7.

Also apparent from Fig. 8 is that a flanged wall (F2) can be made to have a strength and ductility equivalent to a barbell wall (B7) by proper detailing of boundary elements.

Energy Dissipation

The cyclic ductility ratio is not adequate to fully evaluate inelastic performance. Some measure of energy dissipation is also required. Figure 9 shows the approach used for evaluating the test results. The energy dissipated, A1, is considered as a percentage of the linear energy capacity, A2. In this manner, energy dissipated is related to a measure of the amount of energy input to the test specimen. For a particular load cycle, a larger ratio of A1/A2 would indicate a hysteretic loop with less "pinching".

In Fig. 9 the ratio of cumulative A1 to cumulative A2 for the specimens is plotted as a function of the cyclic top deflection ductility ratio. This figure indicates that for equal ductility ratios the percentage of energy dissipated was essentially the same for the specimens tested.

CONCLUSIONS

The following observations are based on the test results:

1. All specimens had a capacity greater than that indicated by the 1971 ACI Building Code (American Concrete Institute, 1971).

2. For walls subjected to maximum shear stresses less than $3\sqrt{f'_c}$ psi ($0.25\sqrt{f'_c}$ MPa), inelastic performance was limited by flexural bar buckling and loss of concrete. Confinement hoops in the boundary elements delayed bar buckling and contained the concrete core. These walls had capacities ranging from 84% to 91% of the calculated maximum monotonic flexural strength.

3. For one rectangular test wall, loss of stability in the compression zone limited inelastic performance.

4. For walls subjected to maximum shear stresses greater than $7\sqrt{f'_c}$ psi ($0.60\sqrt{f'_c}$ MPa), inelastic performance was limited by web crushing. However, the walls did sustain a significant number of inelastic cycles. Compressive axial loads increased strength and ductility. Addition of shear reinforcement did not significantly affect strength or ductility where web crushing was the mode of failure.

5. Deflection ductility of the structural walls increased with decreasing levels of nominal shear stress.

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TABLE 1 - SUMMARY OF TEST SPECIMENS

Specimen	Shape	Axial Load psi	f_c psi	f_y for ρ_f (ksi)	Reinforcement (%)			
					ρ_f	ρ_h	ρ_n	ρ_s
R1		--	6490	74.2	1.47	0.31	0.25	--
R2		--	6735	65.3	4.00	0.31	0.25	2.07
B1		--	7685	65.2	1.11	0.31	0.29	--
B3		--	6860	63.5	1.11	0.31	0.29	1.28
B4 (1)		--	6530	65.3	1.11	0.31	0.29	1.28
B2		--	7775	59.5	3.67	0.63	0.29	--
B5		--	6570	64.4	3.67	0.63	0.29	1.35
B5R (2)		--	6205	--	3.67	0.63	0.29	1.35
B6		423	3165	63.9	3.67	0.63	0.29	0.81
B7		545	7155	66.4	3.67	0.63	0.29	1.35
B8		545	6085	64.9	3.67	1.38	0.29	1.35
F1		--	5575	64.5	3.89	0.71	0.30	--
F2		482	6610	62.4	4.35	0.63	0.31	1.43

(1) Monotonic loading
(2) Repaired specimen
(3) 1000 psi = 1.0 ksi = 6.895 MPa

ρ_f = ratio of main flexural reinforcement area to gross concrete area of boundary element
ρ_h = ratio of horizontal shear reinforcement area to gross concrete area of a vertical section of wall web
ρ_n = ratio of vertical web reinforcement area to gross concrete area of a horizontal section of wall web
ρ_s = ratio of effective volume of confinement reinforcement to the volume of core in accordance with Eq. A.4 of ACI 318-71.

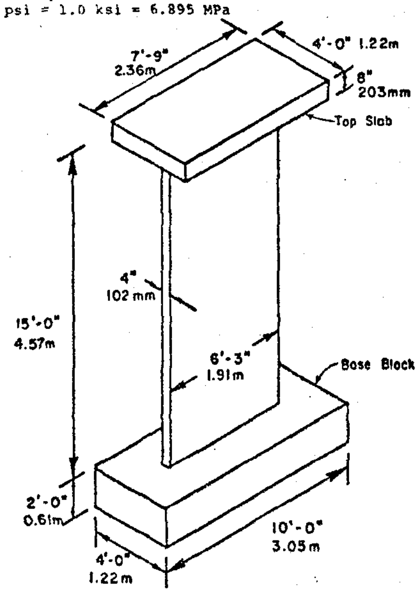

Fig. 1 Nominal Dimensions of Test Specimen with Rectangular Cross Section

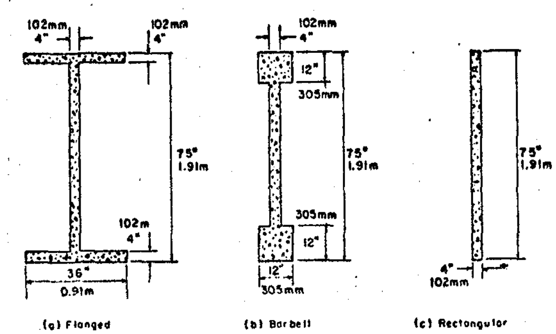

Fig. 2 Nominal Cross-Sectional Dimensions

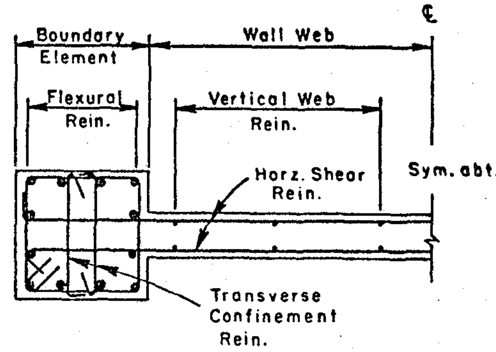

Fig. 3 Wall Section

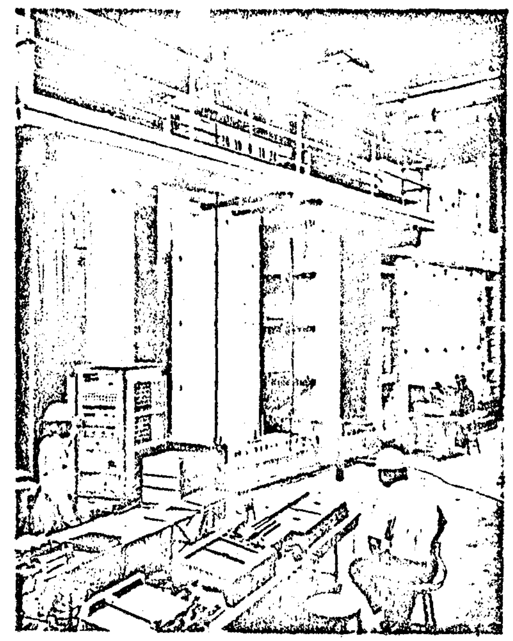

Fig. 4 Test Setup

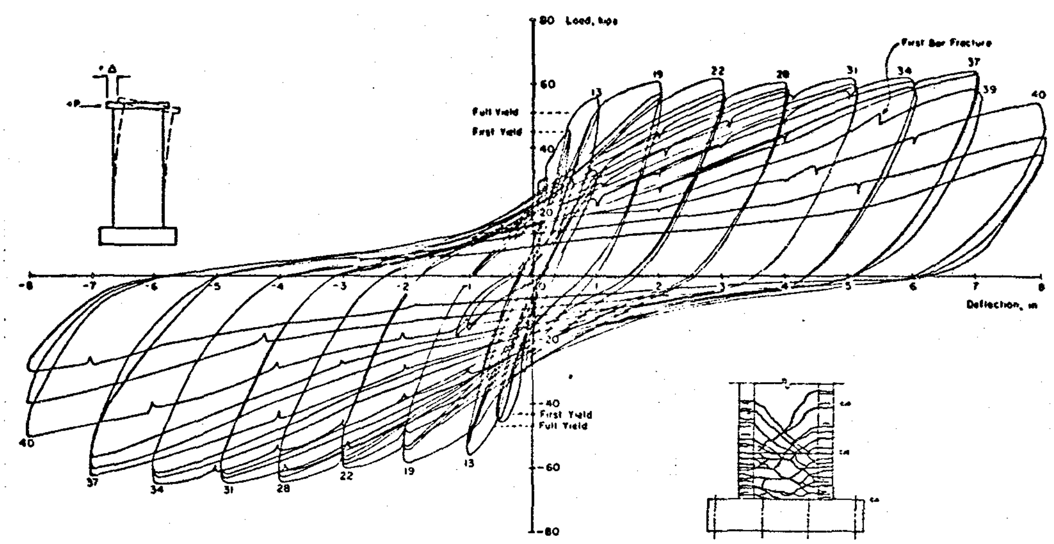

Fig. 5 Load versus Deflection Relationship for Specimen B3 with $v_{max} = 3.1\sqrt{f'_c}$ psi ($0.26\sqrt{f'_c}$ MPa)

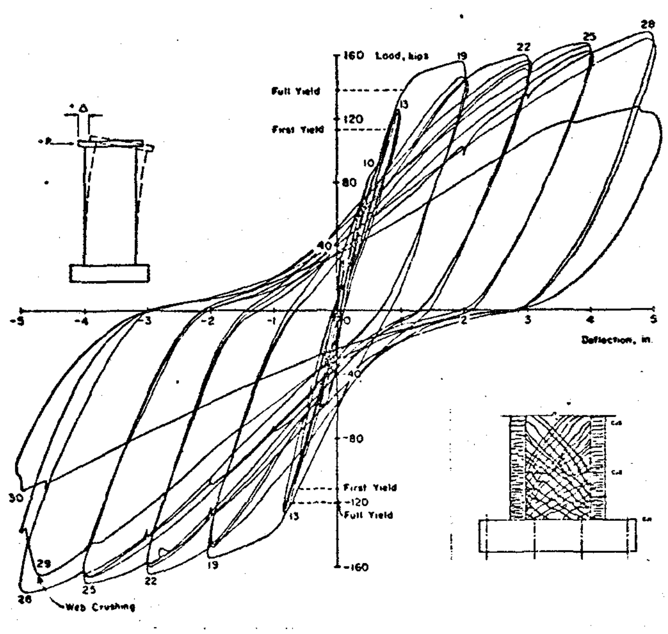

Fig. 6 Load versus Deflection Relationship for Specimen B5 with $v_{max} = 8.8\sqrt{f'_c}$ psi ($0.73\sqrt{f'_c}$ MPa)

TABLE 2 SPECIMEN STRENGTHS

Specimen	Confined Boundary Element	Axial Load psi	ACI Design				Full Yield Load					Maximum Load						Failure Mode (5)
			Flexure		Shear		Calculated(3)		Observed		Obs./Calc.	Calculated(3)		Observed		Obs./Calc	Obs./ACI(4)	
			kips	$\sqrt{f'_c}$(1)	kips	$\sqrt{f'_c}$	kips	$\sqrt{f'_c}$	kips	$\sqrt{f'_c}$		kips	$\sqrt{f'_c}$	kips	$\sqrt{f'_c}$			
R1	No	--	18	0.9	82(2)	4.2	17.7	0.9	21.8	1.1	1.23	29.1	1.5	26.6	1.4	0.91	1.48	F
R2	Yes	--	35	1.8	82(2)	4.2	33.2	1.7	41.8	2.1	1.26	57.3	2.9	48.7	2.5	0.85	1.39	F
B1	No	--	46	2.2	82(2)	3.9	42.9	2.0	51.0	2.4	1.19	72.1	3.4	61.0	2.9	0.85	1.33	F
B3	Yes	--	46	2.3	82(2)	4.1	41.9	2.1	51.5	2.6	1.23	73.4	3.7	62.0	3.1	0.84	1.35	F
B4	Yes	--	46	2.4	82(2)	4.2	43.1	2.2	54.6	2.8	1.27	74.3	3.8	75.3	3.9	1.01	1.64	F
B2	No	--	129	6.1	127	6.0	115.6	5.5	128.0	6.0	1.11	170.9	8.1	152.8	7.2	0.89	1.18	WC
B5	Yes	--	129	6.6	127	6.5	123.1	6.3	138.0	7.1	1.12	213.7	11.0	171.3	8.8	0.80	1.33	WC
B5R	Yes	--	129	6.8	127	6.7	123.1	6.5	--	--	--	213.7	11.5	167.8	8.9	0.79	1.30	WC
B6	Yes	423	157	11.6	132	9.7	154.5	11.4	173.9	12.9	1.13	190.5	14.1	185.5	13.8	0.97	1.41	WC
B7	Yes	545	173	8.5	148	7.3	174.0	8.6	187.5	9.2	1.08	256.2	12.6	220.4	10.9	0.86	1.49	WC
B8	Yes	545	173	9.3	186(6)	9.9	171.6	9.2	189.0	10.1	1.10	241.4	12.9	219.8	11.7	0.91	1.27	WC
F1	No	--	145	8.1	140	7.8	148.1	8.3	150.6	8.4	1.02	242.6	13.5	187.9	10.5	0.77	1.30	WC
F2	Yes	482	170	8.7	148	7.6	164.4	8.4	180.3	9.2	1.10	240.8	12.3	199.5	10.2	0.82	1.34	WC

(1) Lateral load in terms of nominal shear stress $v = \dfrac{V}{0.8\ \ell_w\ b\sqrt{f'_c}}$ (psi)

(2) Shear reinforcement governed by maximum bar spacing

(3) Calculated monotonic flexural strength from analysis based on strain compatibiity using measured material properties including strain hardening of reinforcement

(4) ACI taken as the lower of flexure or shear design strength

(5) F = Flexure, WC = Web Crushing

(6) Maximum $v = 10\sqrt{f'_c}$ governs, ACI Design Shear for B8 would be 256 kips = $13.7\sqrt{f'_c}$ disregarding the maximum allowable

(7) 1 kip = 4.448 kN, $1.0\sqrt{f'_c}$ (psi) = $0.08304\sqrt{f'_c}$ (MPa)

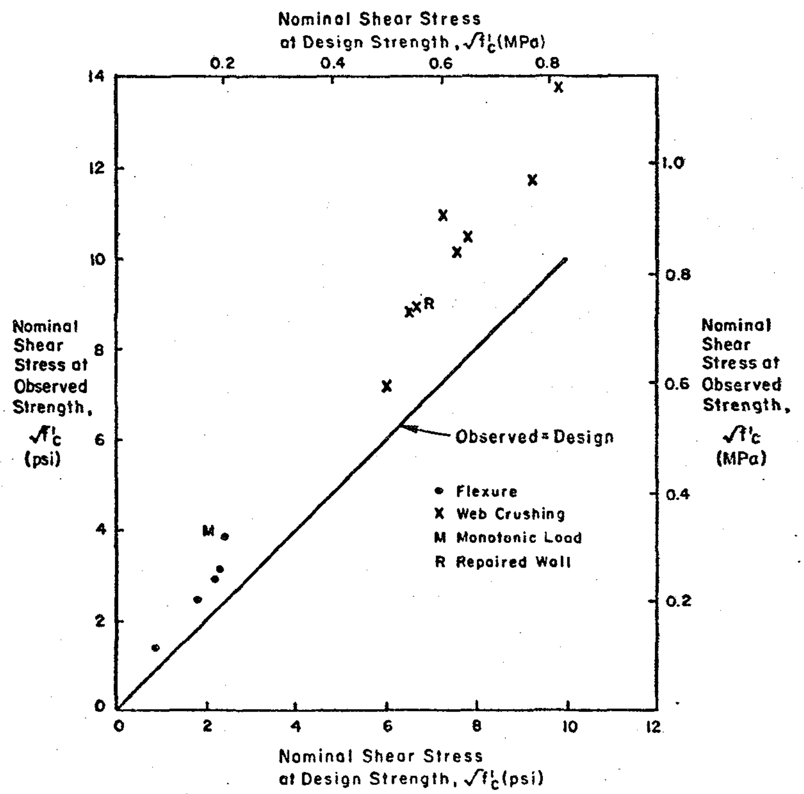

Fig. 7 Observed Strength versus Design Strength

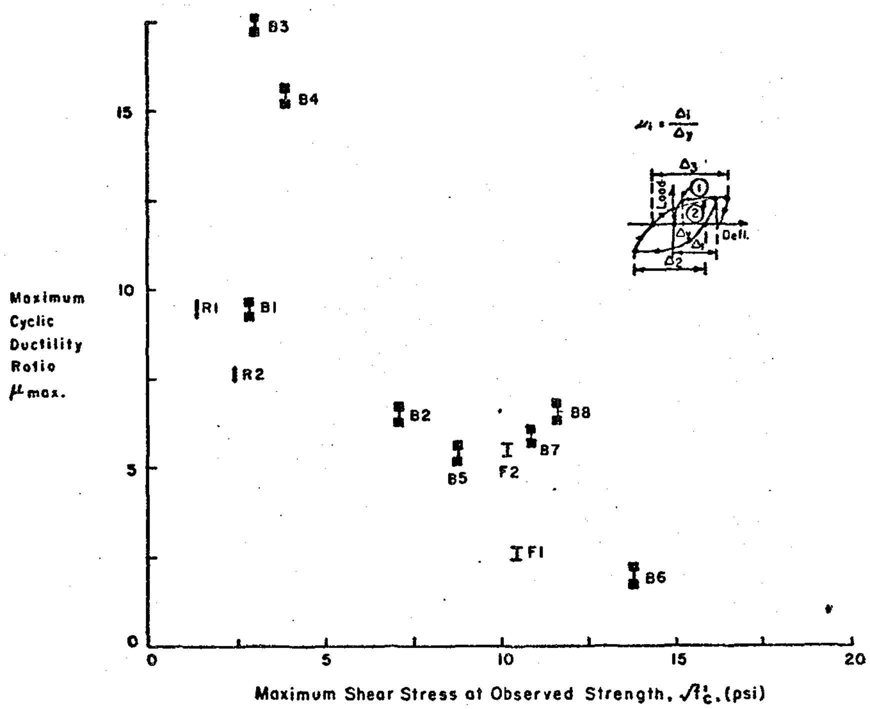

Fig. 8 Maximum Cyclic Deflection Ductility versus Maximum Nominal Shear Stress in Wall

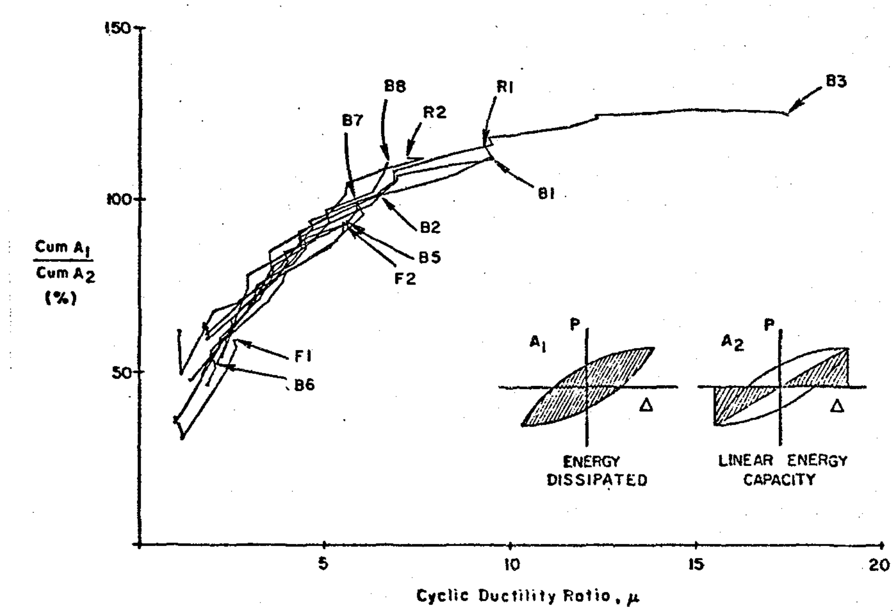

Fig. 9 Ratio of Dissipated Energy to Linear Energy Capacity versus Cyclic Deflection Ductility Ratio