SEISMIC DESIGN IMPLICATIONS OF HYSTERETIC BEHAVIOR OF REINFORCED CONCRETE STRUCTURAL WALLS

by

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SYNOPSIS

After reviewing current code aseismic design provisions for R/C structural wall systems and present knowledge of their hysteretic behavior, research being conducted on this subject at Berkeley is summarized. The facility selected for testing frame-wall subassemblages, details of the 1/3-scale wall subassemblage models tested, the fabrication procedure, mechanical characteristics of the materials, and the selection of the test loadings for these models are discussed. The main results obtained are evaluated and their implications for aseismic design of frame-wall structural systems are discussed.

INTRODUCTION

In buildings with frame-wall structural systems, the interaction between wall and frame, particularly during their hysteretic behavior under severe earthquake-like conditions, is not very well understood at present. While the UBC and SEAOC recommend increasing the value of earthquake forces in calculating shear stresses in shear walls of buildings without a 100% moment-resisting frame, the ACI does not. Although it is convenient to have a greater safety factor against nonductile shear failures, it is not believed that merely increasing the value of the design seismic loads is the best way of achieving this. To achieve ductile hysteretic behavior, a wall should be designed against the maximum shear that can be developed according to the actual flexural capacity (as affected by the axial force) of its critical region and considering the critical moment-shear ratio that can exist at such a region. Even if the maximum shear can be estimated with sufficient engineering accuracy, there still remains the problem of designing against it. Up until 1970, most of the available experimental results on the behavior of wall elements were obtained from tests of one- or two-story R/C walls, or infilled R/C frames which were subjected to simplified loading conditions which did not simulate the actual effects of earthquake excitations. Only in some recent investigations[1,2,3] have attempts been made to simulate the loading conditions expected in slender flexural walls subjected to earthquake excitations. Need for improvements in predicting the mechanical behavior of wall systems has led to the initiation of the investigation partially described herein.

<u>Objectives and Scope</u>. - The ultimate objective of this investigation is to develop practical methods for the aseismic design of combined frame-wall structural systems. To achieve this objective, integrated analytical and experimental studies are being conducted.[4] In this paper emphasis is placed on the discussion of experimental studies. The main objective of these studies is to obtain reliable data regarding the linear and nonlinear (particularly hysteretic) behavior of such wall systems. Only those analytical results needed for planning the design of the testing facility and specimen loading programs, and for judging the possible aseismic design implications of the behavior observed in the experiments will be briefly discussed.

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SELECTION OF TYPE, DESIGN AND CONSTRUCTION OF TESTING FACILITY

To investigate in detail the mechanical behavior of frame-wall systems, it was decided to develop a loading facility capable of simulating earthquake effects rather than to use the existing shaking table. For economic reasons, it was decided to test significant subassemblages of the structural system rather than large-scale models of entire buildings. Predicting the in-plane seismic behavior of frame-wall systems requires information on the variation of the lateral shear-displacement relationship for each story (Fig. 1). To simulate the actual boundary conditions of a particular story, subassemblages of at least two or three stories have to be tested (Fig. 1).(4)

Two buildings, ten (Fig. 1)- and 20-stories, 61 ft x 180 ft each, were designed according to present UBC provisions. From analyses of the response of these buildings to different ground motions, it was possible to estimate the relative intensity of the forces acting on the bottom three-story subassemblages. The results led to the design of a facility capable of testing 1/3-scale models of this type of subassemblage. The principal feature of this facility is its ability to simulate pseudo-statically the dynamic loading conditions which could be induced in subassemblages of buildings during earthquake ground shaking (Fig. 2). Assuming that the wall alone resists most of the lateral inertial forces, it would not only be necessary to apply lateral forces, but also, forces which would simulate the effect of overturning moments and gravity loads existing above the top floor of the subassemblages [Fig. 2(c)]. Therefore, to simulate the actual inelastic behavior of this subassemblage when it forms part of the whole wall, the synchronized shear, overturning, and axial forces must be applied simultaneously.

The walls are tested in a horizontal position (Fig. 3). The testing facility consists of a set of reaction blocks, loading devices, ancillary devices, instrumentation and data acquisition systems.(4) Specimens from 1/2- to 1/4-scale of two- to four-story prototype subassemblages can be tested in this facility.

FABRICATION AND MATERIAL MECHANICAL CHARACTERISTICS OF TEST SPECIMENS

A series of tests have been conducted on four 1/3-scale wall component models of the bottom three stories of the ten-story frame-wall system shown in Fig. 1. The four specimens consisted of a 4-in. thick wall framed by two 10-in. sq. columns and a portion of 3-in. thick floor slabs (Fig. 4). The only difference between the four specimens was in the way that the concrete of the edge members was confined. While spirals were used in specimens 1 and 2 (series 1), Fig. 4(a), square ties were used in specimens 3 and 4 (series 2), Fig. 4(c). To simulate the construction work in the field, the specimens were cast story by story in their vertical position. The two specimens of each series were cast simultaneously to minimize variation in the mechanical characteristics of the concrete. The material mechanical characteristics are summarized in Table 1 which shows that the actual concrete compressive strength and yielding strengths of the reinforcing bars were considerably higher than those specified.

LOADING CONDITIONS AND TESTING PROCEDURE

Loading Conditions. - Rather than simulate the critical load combinations of gravity (dead and live) and seismic loads as specified by the 1973 UBC, it was decided to investigate the behavior of these code designed walls under the most critical load combination which could be developed in the case of an extreme earthquake ground shaking. Table 2 illustrates the differences in some of the loading conditions that were derived for the wall model of the prototype of Fig. 1 using different methods for evaluating the seismic forces. The considerable discrepancies between the resulting shear span values point out not only the difficulties in selecting the critical combination of inertial forces, but also, the need for carefully interpreting results obtained in experimental investigations in terms of the actual seismic behavior of structures. Specimens were tested under the load combination corresponding to the last case presented in Table 2.

Testing Procedure. - The two axial forces necessary for simulating the effects of gravity forces were applied first and were the same for the four specimens. The effects of seismic forces were introduced following a different loading pattern in each of the specimens tested. The four specimens were first subjected to cycles of full seismic force reversals in the working load range. In walls 1 and 3, the lateral force and change in column axial forces needed to reproduce the corresponding change in overturning moment were supposed to increase monotonically until a reduction in the lateral resistance could be observed. In the test of wall 1, however, a cycle with significant inelastic displacement reversal was introduced long before the drop in lateral resistance (Fig. 5). Walls 2 and 4 were subjected to a history of lateral shear and corresponding overturning moment that induced gradually increasing cycles of full reversal lateral displacement with at least three cycles at each displacement amplitude (Fig. 6).

TEST RESULTS AND THEIR EVALUATION

Overall Response. - Figures 5 and 6 are composite graphs illustrating the overall responses for the four specimens tested. These graphs facilitate evalution of the two main variables of the tests reported herein, i.e. the effect of (1) cycling with reversal deformations versus monotonically increasing loads; and (2) different ways of confining concrete of edge members.

Cycling with Displacement Reversals vs. Monotonic Loading. - The curves obtained under monotonically increased loading for walls 1 and 3 provide approximate envelopes for the hysteretic behavior obtained for walls 2 and 4, respectively (Figs. 5 and 6). Before any significant reduction in strength was observed, wall 3 deformed up to nearly 7 in., giving a displacement ductility, $\mu_\delta = \delta/\delta_y$, of about 10. However, when the load was reversed, the wall buckled under a lateral load of only 80 kips. With wall 1 (which was subjected to only one significant cycle of reversal of inelastic deformation), the maximum μ_δ was 6.1, i.e. a reduction of about 40%; and with walls 2 and 4, the maximum μ_δ was 4.2, i.e. a reduction of nearly 60%. It can therefore be concluded that while repeated reversals of lateral loads did not affect the strength of the wall, they did reduce the ductility by as much as 60%. It should be noted, however, that the maximum cyclic ductility, $\mu_{\delta_{cyc}}$, for walls 2 and 4 ($\doteq$ 7.5) was only 25% less than the μ_δ of wall 3. Analysis of the hysteretic loops for walls 2 and 4 (Fig. 6) indicates that each time the absolute value of peak deformation was increased, there was a degradation in the initial stiffness and energy dissipated during the following cycle. Although there was a loss in strength with repetition of cycles at same tip deflection, this was noticeable only after the first cycle and it stabilized at the third cycle except when the tip deflection reached a value of 2.8 in. (where crushing of the wall panel was observed). At this displacement, no stabilization of the hysteretic loop was obtained.

Different Ways of Confining Column Concrete. - From comparisor of the curves for walls 2 and 4 (Fig. 6) it can be concluded that the hysteretic behavior of these two walls were very similar except for the following differences: (1) for the same tip deflection the resistance of wall 2 was greater than that of wall 4; and (2) there was slightly less stiffness degradation and pinching in the curves of wall 4. These two observed differences are interrelated and consistent because the larger the shear developed in a cycle, the larger the stiffness degradation and pinching during the next cycle. However, these differences are not a consequence of the differences in the concrete confinement of columns. The reason for the smaller resistance of wall 4 is the lower strength at yielding and in the strain-hardening range of the #6 bars used in the columns when compared with those of wall 2 (see Table 1). Thus, it can be concluded that the closely spaced square ties were as effective as the spirals. The only noticeable difference was observed in the pattern of failure of the columns after the wall panel failed: the columns of wall 4 were bent (kinked) into a double curvature along a length smaller than that of wall 2, with more pronounced buckling of its main reinforcement and crushing and spalling of its confined concrete in this region.

Contribution of Different Sources of Deformation: The analysis of data recorded permitted study of the contributions of: (1) flexural deformation; (2) shear deformations at each story; (3) slippage along the construction joints; and (4) fixed-end rotation due to slippage of the reinforcement along its embedment length at the foundation. As illustrated in Fig. 7, for the walls subjected to cyclic loading, the first two sources of deformation were the most significant. Up to first yielding, the shear deformation at each story was similar, the first story deformation being somewhat higher because of the greater amount of cracking. After yielding, most of the shear deformation was concentrated in the first story. Furthermore, at a total tip displacement of 1.4 in., there was already a considerable increase in shear deformation of the first story upon repetition of cycles having the same peak displacement (compare Figs. 6 and 8). At a tip displacement of 2.8 in., the shear deformation was concentrated in the lower part of the first story where the concrete crushed and spalled along a horizontal band (Fig. 9). As a consequence of this type of wall panel failure, the columns began deflecting in a double curvature chape, leading to the failure mechanism shown in Fig. 9.

ASEISMIC DESIGN IMPLICATIONS OF EXPERIMENTAL RESULTS

Despite the limited amount of specimens tested, analysis of the data obtained enables the following observations to be formulated. These observations, however, should be evaluated as tentative and subject to modification as more data become available: (1) It is possible to design structural wall components capable of developing large ductilities even when subjected to full deformational reversals inducing nominal unit shear stresses up to $10\sqrt{f'_c}$. Although the μ_δ under monotonic loading reached a value of 10, this large μ_δ should not be used for design since its development can lead to instability of the wall under loading reversal. (2) Although the μ_δ was reduced significantly due to full reversals, from 10 to about 4, it should be noted that this reduced μ_δ corresponded to a $\mu_{\delta_{cyc}}$ of 7 and that it can be considered large enough to permit the development of energy absorption and energy dissipation capacities exceeding even those that would be demanded in the case of very severe earthquake shaking. Furthermore, at this reduced μ_δ the confined core of the columns remained sound and capable of resisting both the effects of axial forces imposed by gravity loads and by lateral loads in the working load range. (3) Present code specifications for design forces, load factors, and design and detailing of critical regions can lead to a wall design which considerably underestimates the amount of shear that can actually develop. The design of flexural walls against shear should be based on the maximum shear that can be developed according to the flexural capacity of the critical region, and on the largest possible shear/bending moment ratio (V_B/M_B) according to the expected dynamic response of the entire building to severe ground motions of different dynamic characteristics. After the tests, several nonlinear dynamic analyses assuming an infinitely ductile model of the prototype were carried out according to the experimentally obtained stiffnesses and strengths of the walls. The results obtained reveal that the shear force in the first two stories of the wall exceeded its shear capacity before the base moment reached the moment capacity of the wall, and that the critical ratio, V_B/M_B, was about 1.46 times the larger value obtained in the elastic analyses. Thus, the shear and overturning moment ratio used in the experiments was unconservative.

ACKNOWLEDGEMENTS

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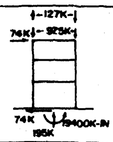
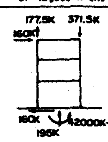
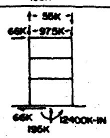
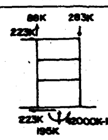
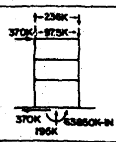
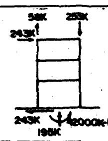
TABLE 1 MECHANICAL CHARACTERISTICS OF MATERIALS

CHARACTERISTICS		AVERAGE STRENGTH AT TIME OF TESTING Walls 1&2	Walls 3&4
Concrete Compressive Strength * (psi)	1st Fl.	5,340	5,100
	2nd Fl.	5,120	5,130
	3rd Fl.	4,770	4,900
Concrete Splitting Tensile Strength (1st Fl.) (psi)		495	480
Concrete Flexural Tensile Strength (1st Fl.) (psi)		645	632
Wall Steel ** (#2 Bars) (psi)	f_y	73,400	73,400
	f_{max}	105,800	105,800
Col. Long. Steel ** (#6 Bars) (psi)	f_y	72,700	64,400
	f_{max}	106,000	92,740
Col. Transverse Steel ** (0.21" ϕ) (psi)	f_y	82,000	63,750
	f_{max}	101,000	69,500

*The specified design strength of concrete was 4000 psi at 28 days.

**The specified yield strength of steel was 60,000 psi.

TABLE 2 COMPARISON OF LOADING CONDITIONS FOR MODEL DERIVED FROM ANALYZING PROTOTYPE STRUCTURES USING DIFFERENT SEISMIC ANALYSIS METHODS

DESIGN CRITERIA	METHODS OF DETERMINING SEISMIC FORCES		SIMPLIFIED LOADING CONDITIONS FOR WALL SUBASSEMBLAGE MODEL: COMPUTED FORCES	ULTIMATE FORCES BASED ON ESTIMATED FLEXURE STRENGTH OF 42,000 K-IN.	SHEAR SPAN $\frac{a(IN)}{(\frac{a}{d})}$
WALLS ALONE RESIST TOTAL SEISMIC LATERAL FORCES	UBC				263 (2.8)
FRAMES AND WALLS RESIST TOTAL SEISMIC LATERAL FORCES	UBC				189 (2.0)
	LINEAR ELASTIC RESPONSE SPECTRUM $\ddot{U}_g = 0.33g$ and $\xi = 5\%$	FIRST THREE MODES			173 (1.9)

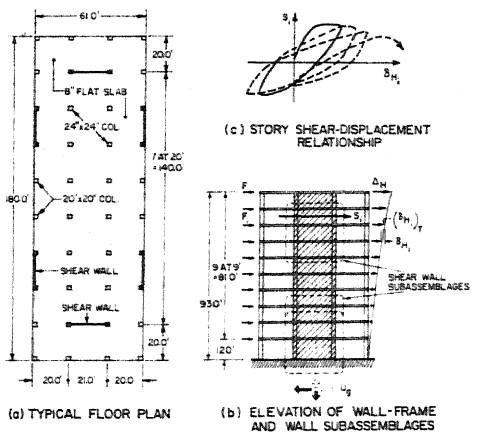


FIG. 1 PROTOTYPE BUILDING, WALL SUBASSEMBLAGES AND STORY SHEAR - DISPLACEMENT RELATIONSHIP

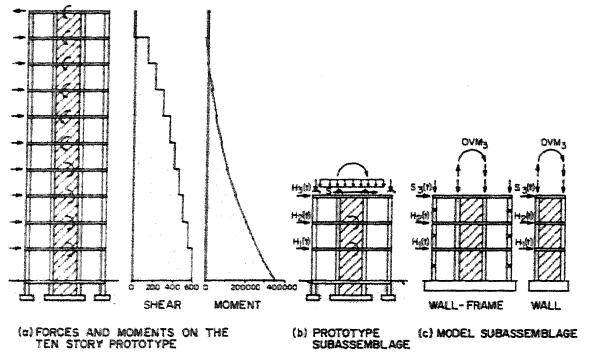


FIG. 2 COMPARISON OF ACTUAL AND SIMULATED LOADING AND SUPPORT CONDITIONS

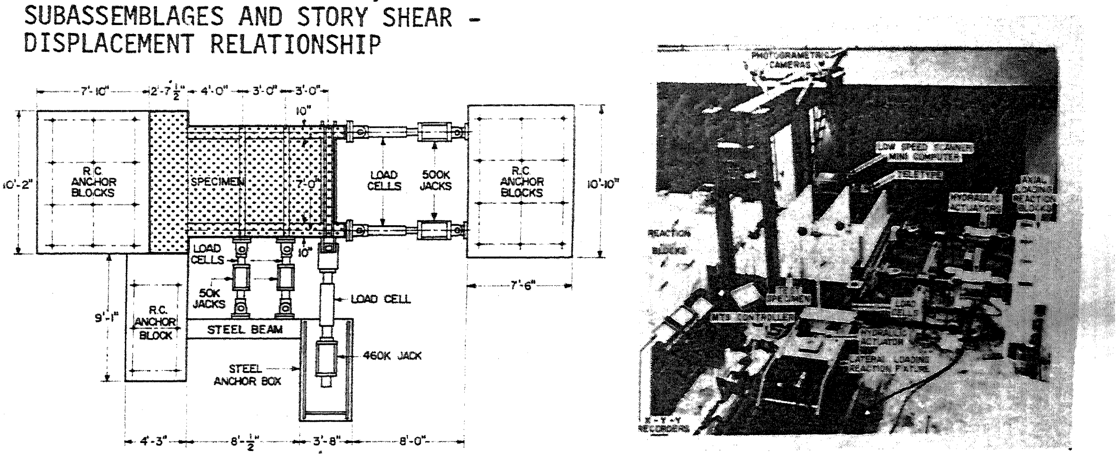
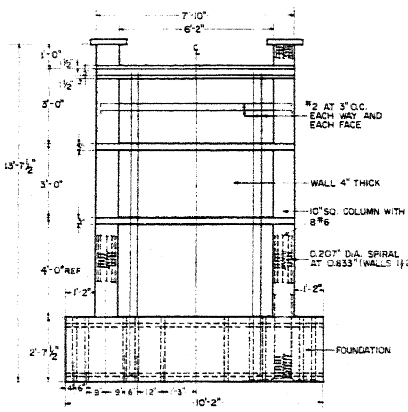
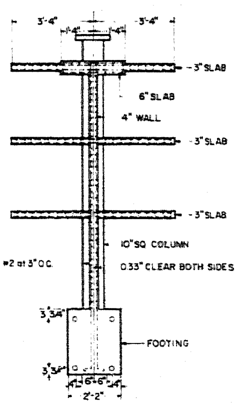


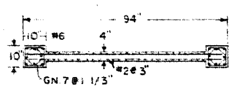
FIG. 3 PLAN AND GENERAL VIEW OF TESTING FACILITY



(a) Elevation of Walls 1 & 2



(b) Vertical Cross-section



(c) Horizontal Cross-section of Walls 3 & 4

FIG. 4 DIMENSIONS AND DETAILS OF WALL SPECIMENS

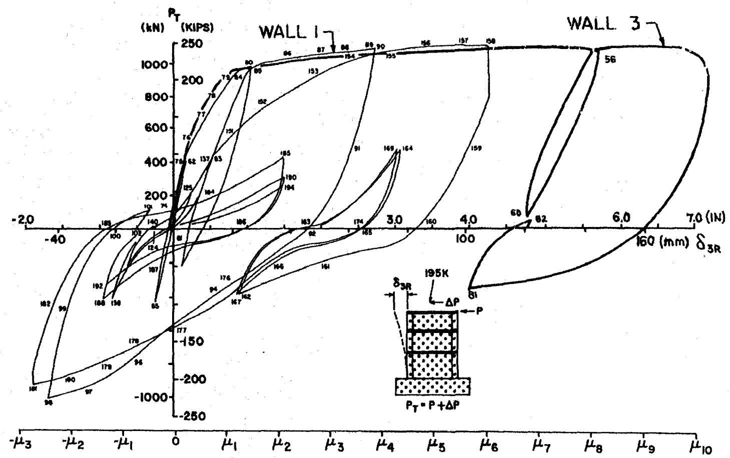


FIG. 5 P_T-δ_{3R} DIAGRAMS - WALLS 1 & 3

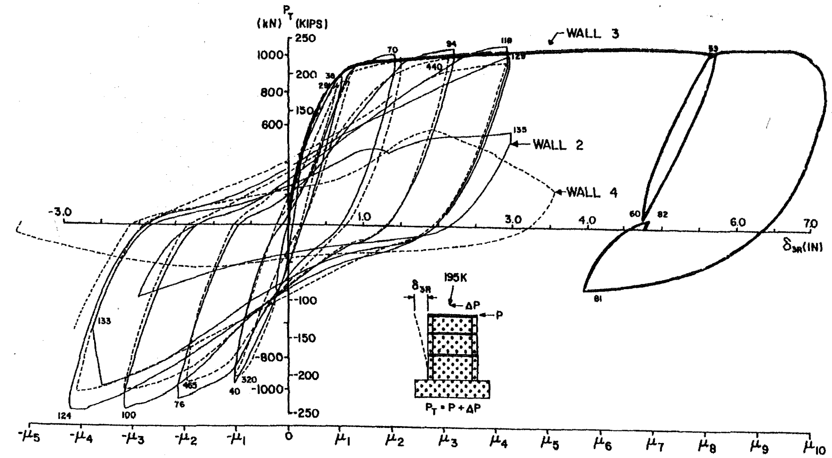


FIG. 6 P_T-δ_{3R} DIAGRAMS - WALLS 2, 3 & 4

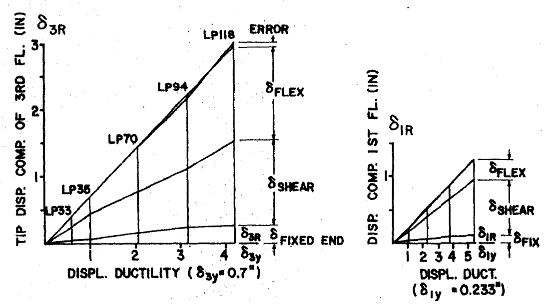


FIG. 7 CONTRIBUTION OF DIFFERENT SOURCES OF DEFORMATION

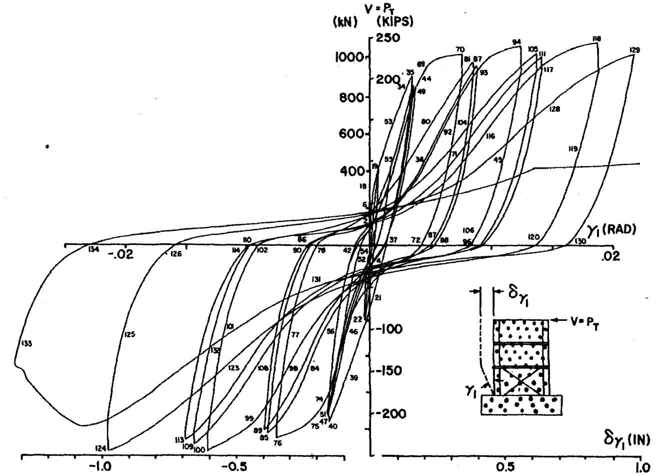


FIG. 8 P_T-γ_1 DIAGRAM - WALL 2

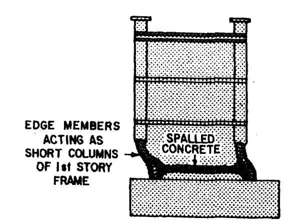


FIG. 9 MECHANISM OF FAILURE OF WALL SPECIMENS