SEISMIC BEHAVIOR OF R/C WALL STRUCTURAL SYSTEMS

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SUMMARY

The state-of-the-practice in the design and construction of R/C ductile moment-resistant space frame buildings is compared with that of shear wall buildings, and it is concluded that frame-coupled wall systems offer great potential for seismic-resistant building construction. Experimental and analytical research results on seismic behavior of isolated walls and coupling girders indicate excellent hysteretic behavior. Problems in designing coupled wall systems are examined. Recommendations are formulated for research to realize the great potential of these systems.

INTRODUCTION

General Remarks. Analysis of the 1976 UBC [1] shows that, of buildings having the same dynamic characteristics, those with shear walls must be designed for seismic forces about 20% to 100% higher than those having ductile, moment-resisting space frames (DMRSF). Furthermore, the UBC design provisions for the shear walls against shear are inconsistent with the philosophy of the design for shear of members of DMRSF. As a consequence of the excellent performance of shear wall structures in recent destructive earthquakes, the following questions have been raised: Is there any new information to justify the modifications of these high seismic forces and the apparent inconsistency in requirements for shear design? Also, can a building whose structural system is based on use of walls be designed with the same distribution of inertial forces along its height as a building whose structural system is a DMRSF, as specified in the present UBC? These questions were the motives for an investigation that began several years ago at Berkeley [2,3,4] with the ultimate objectives described below and which motivated this paper.

Objectives. The ultimate objective of the investigation being conducted at Berkeley is to develop practical methods for the seismic design of combined wall-frame structural systems. The specific objectives of this paper are to summarize the state-of-the-practice and state-of-the-art in predicting seismic behavior and in the design and construction of R/C structural wall systems, to ascertain whether there is sufficient data to recommend modifications in current practice, to analyze problems that still remain without satisfactory solutions, and to formulate research needs to solve them.

STATE-OF-THE-PRACTICE OF SEISMIC-RESISTANT DESIGN OF SHEAR-WALL SYSTEMS

Introductory Remarks. To recognize problems that are still present in the design of seismic-resistant buildings having shear wall structural systems in comparison with the design of DMRSF buildings, it is convenient to analyze all the main steps that are involved in trying to satisfy the basic design equation, i.e., the DEMAND (on stiffness, strength, stability, durability, ductility, and energy absorption and energy dissipation capacity) shall be exceeded by the SUPPLY.

Estimation of Demands. It has been shown [5-8] that the major uncertainties in the whole design process are involved in this estimation, usually

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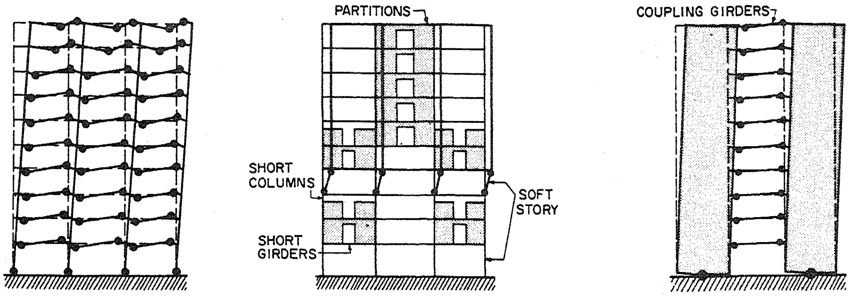
conducted through numerical analysis, due to the difficulties in predicting what the critical seismic loading would be during the service life of the structure (proper establishment of design earthquakes) and the state of the building when the critical seismic ground motion at the site of the building occurs (proper selection of the building model that should be analyzed).

1. Seismic Loading. Present seismic code [1] defines this loading by specifying the value of the total lateral force or shear at the base, V, and its distribution over the height of the building. According to this code, shear wall buildings must be designed for a total V 20% to 100% higher than a DMRSF building having the same fundamental period of vibration, T. Why are R/C wall structural systems penalized? Usually it is claimed that R/C walls lack ductility. As will be shown later, properly designed and constructed R/C walls can develop large ductility and, even more importantly, dissipate large amounts of energy through stable hysteretic behavior. Furthermore, since the modeling of a building having as a structural system a proper layout of walls (particularly coupled walls) offers considerably less uncertainty than a DMRSF building (due largely to the sensitivity of the latter to the interacting effects of the nonstructural elements), it seems illogical to require that a properly designed R/C wall building be designed for higher loads that DMRSF buildings. The ATC [9] recognizes, through a response modification coefficient R, that properly designed and constructed walls can dissipate more energy than a DMRSF. While for an R/C special moment frame, which is a DMRSF, R is specified as 7, for a dual system based on use of R/C shear walls, R is 8, i.e., the seismic forces for which it should be designed are about 15% less than those of the DMRSF having the same dynamic characteristics. Regarding the distribution of the lateral force over the height of the structure, the UBC [1] specifies just one set of formulas for all buildings no matter what structural system is used. While the code distribution appears justified for those types of structures in which the effects of moments (overturning moment) are controlling the design, it does not seem to be a conservative pattern for cases in which shear can be a problem, as it is in the case of R/C walls.

2. Modeling. A design can only be effective if it can be constructed, i.e., the model used for conducting the estimation of the demands in the design process should be realistic. An analysis of the uncertainties that exist in developing realistic models for a R/C DMRSF vs. a R/C shear wall building indicates that the uncertainties involved in the first are larger than in the second. The main reasons follow: (1) *Effect of Higher Modes and Inelastic Moment Redistribution on Actual Response of a Building*--The design and construction of a DMRSF building is based on the philosophy of strong columns-weak girders. Column hinging should be allowed at the base of the bottom columns only after all the girders' plastic hinges have been developed (Fig. 1a). Although this requirement can be satisfied by designing the columns to have a flexural strength larger than that required at the joint considering the ultimate flexural capacity of the girders (as it is specified in the codes), in reality due to effects of higher modes and unequal distribution of beam input moments at a joint between the column above and below the joint, early and significant column hinging can develop. To avoid this, Paulay in Ref. 5 has suggested that columns be designed to resist the moment computed according to the girder capacity by a dynamic moment magnification factor which is a function of the computed fundamental period, T, and can vary from 1.2 to 1.8. It is obvious, then, that while the actual behavior of a DMRSF can lead to early formation of column hinging, this could not happen in coupled wall systems, since the walls act as a column having a flexural strength considerably higher than that of the

girders. Thus, the development of inelastic deformation at the girder can be controlled with higher reliability in coupled wall structural systems than in DMRSF systems. (2) *Effect of Nonstructural Elements.* The seismic response of DMRSF systems is considerably more sensitive to the effects of nonstructural elements than wall systems. The effects can be grouped in two categories: (a) walls, partitions, stairways, etc. can considerably change the dynamic characteristics of the whole DMRSF system, particularly increasing the fundamental period and modifying the torsional response of the building; and (b) these walls, partitions, stairways, etc. can create "soft story" and/or "short columns and/or girders" in the DMRSF, as illustrated in Fig. 1. The effect of such nonstructural elements will be considerably less in the case of coupled wall structural systems.



a. DMRSF: Assumed b. DMRSF: Effects of Nonstructural Elements c. Coupled Walls

FIG. 1. COMPARISON OF BEHAVIOR OF DMRSF AND COUPLED WALLS

Concluding Remarks. Use of coupled walls in seismic-resistant design seems to have great potential. To realize this potential it would be necessary to prove that it is possible to design and construct "ductile coupling girders" and "ductile walls" that can SUPPLY the required strength, stiffness, and stability and dissipate significant amounts of energy through stable hysteretic behavior of their critical regions. A review of present knowledge in these matters is presented below.

STATE-OF-THE-ART OF SEISMIC BEHAVIOR OF SHEAR-WALL SYSTEMS

Introductory Remarks. The state-of-the-art in seismic behavior of shear-wall structural systems in several countries, particularly Japan, New Zealand and the U.S., up to 1977 has been discussed in detail in Ref. 5. The experimental and analytical studies presented in this reference, as well as those studies carried on to date, have been reviewed by the author in light of the previous discussion, i.e., to ascertain whether it is possible to design and construct shear walls and coupling girders with sufficient energy dissipation capacity to permit the construction of efficient seismic-resistant frame-coupled wall structural systems. The main observations obtained from this review follow.

Isolated Walls. The behavior of walls under loading histories simulating those that can develop during the response of a shear wall building to severe seismic ground motions has recently been extensively studied, experimentally and analytically, particularly at the laboratories of the Portland Cement Association (PCA) [10] and of the University of California, Berkeley [2-4]. A total of 34 experiments have been conducted (16 at PCA and 18 at Berkeley) to study the effects of several parameters on the hysteretic behavior of these walls. A brief discussion of the effects of some of these parameters follows.

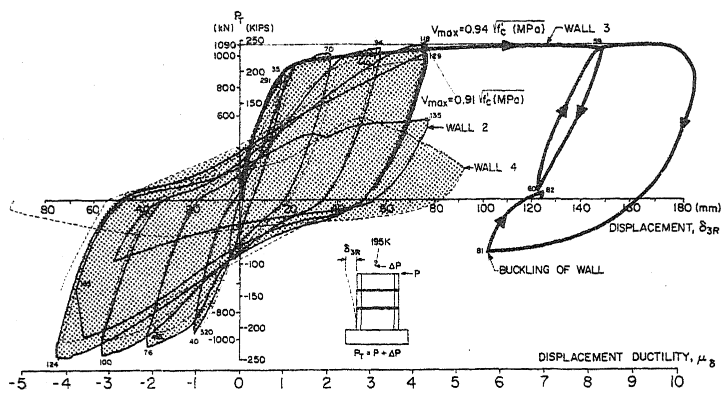


FIG. 2. R/C WALLS: COMPARISON OF BEHAVIOR UNDER MONOTONICALLY INCREASING LOADING (WALL 3) WITH HYSTERETIC BEHAVIOR UNDER INCREASING DISPLACEMENT REVERSALS (WALLS 2 AND 4).

1. Loading History. A larger displacement ductility is obtained when the wall is subjected to monotonically increasing lateral load. As can be seen in Fig. 2, before any significant reduction in strength was observed (1090 KN), wall 3 deformed up to nearly 180 mm, giving a displacement ductility, μ_δ, of 10 (the μ_δ of the first story was 14). However, when the load was reversed, the wall buckled under a lateral load of only 356 KN. Therefore, this large displacement ductility cannot be used for seismic-resistant design. Stability under load reversal can control the maximum ductility that can be used. The main effects of cyclic loading inducing reversals of loads are to reduce the μ_δ and originate a degradation in the initial stiffness (pinching of the hysteretic loops). The larger the deformation reversals, the larger the reductions. For example, cycling under full deformation reversals reduces the μ_δ from 10 to about 4 (which corresponds to a cyclic ductility ratio of about 7), and the initial stiffness during reloading is reduced so drastically that the energy dissipated in one cycle is only about 50% of that which will result if the hysteretic loop is that of an elasto-perfectly plastic type. However, in spite of these reductions, the total amount of energy dissipated by walls 2 and 4, which were subjected to cyclic loading with full deformation reversals, was more than three times that dissipated by the walls subjected to monotonically increasing loads, i.e., wall 3, Fig. 2. Furthermore, at the reduced $\mu_\delta = 4$ and after the wall panel failed completely, the edge members of the barbell cross-section walls remained sound and capable of resisting the effects of the axial forces imposed by the gravity loads combined with the effect of lateral loads at working load level.

2. Cross-Section Type: Barbell vs. Rectangular. Figures 3 and 4 compare the behavior of these two types of cross sections under monotonic and cyclic loading. The better behavior of the barbell is clear from these figures. The main reason is the earlier lateral buckling of the rectangular with respect to the barbell, due to the smaller thickness of the rectangular wall (114 mm) with respect to the thickness of the barbell edge member (254 mm). Spalling of the concrete cover in the rectangular wall results in a 48% reduction in the out-of-plane stiffness, leading to its failure by out-of-plane buckling.

3. Edge Member Confinement. Three different types of lateral reinforcement

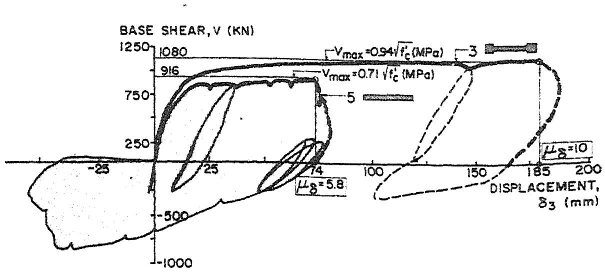


FIG. 3. BARBELL VS. RECTANGULAR CROSS-SECTION WALLS: COMPARISON OF BEHAVIOR UNDER MONOTONICALLY INCREASING LOAD

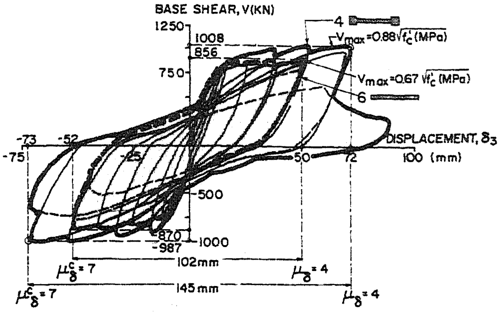


FIG. 4. BARBELL VS. RECTANGULAR CROSS-SECTION WALLS: COMPARISON OF BEHAVIOR UNDER CYCLIC LOADING

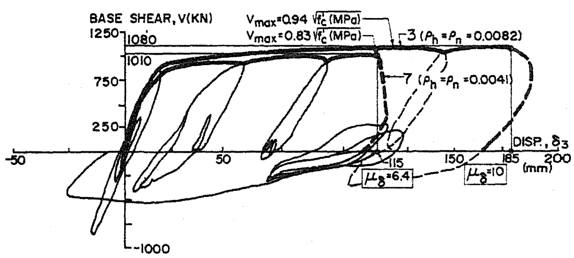


FIG. 5. BARBELL CROSS-SECTION WALLS: EFFECT OF AMOUNT OF WALL-PANEL REINFORCEMENT

were studied: (1) circular spiral; (2) square ties; and (3) rectangular ties. Confinement and overall behavior of the edge members where spiral and square ties were used were better than that obtained with rectangular ties. Protection against buckling of the longitudinal reinforcement offered by the rectangular ties was less effective than that offered by the square ties; the best protection was that offered by the spiral.

4. Wall Reinforcement: Amount and Arrangement. *Amount, ρ:* The strength capacity is practically unaffected by the amount of wall reinforcement (Fig. 5). The larger the amount and particularly the closer the spacing of the wall panel reinforcement, the more ductile the behavior. However, the degree of improvement is not in direct proportion to ρ but is significantly smaller. The plots of Fig. 5 illustrate these observations, although it should be noted that wall 7 was submitted to a series of cycles with full deformation at yielding level which could affect (decrease) the actual μ_δ under monotonically increasing loads [4]. *Arrangement:* Diagonal arrangement of the reinforcement (i.e., inclined at 45°) results in better behavior than the vertical and horizontal reinforcing bar arrangement (Fig. 6).

5. Shear Stress, v. The nominal shear stress, v, is usually computed as V/hd, where h is the thickness of the wall panel and d is the effective depth between the extreme compression fiber and the centroid of the rebars in tension. Code requirements allow use of the value 0.8 ℓ_w as d, where ℓ_w is the total length of the wall. Using this last value, the v_{max} in the tests carried out by the PCA varied from 0.12 to $1.15\sqrt{f'_c}$(MPa). On the other hand, during the 18 tests conducted at Berkeley, this value ranged from 0.71 to $1.12\sqrt{f'_c}$(MPa). Analyses of

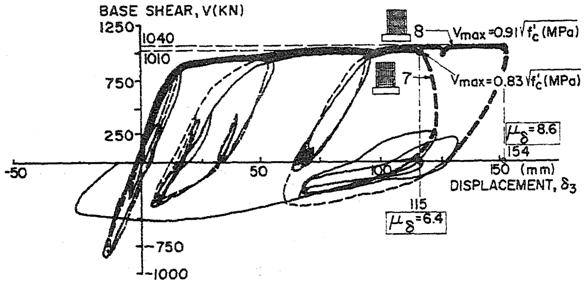


FIG. 6. BARBELL CROSS-SECTION WALLS: EFFECT OF WALL-PANEL REINFORCEMENT ARRANGEMENT

these results indicate that for similar walls the smaller the value of v_{max}, the better the overall behavior under monotonically, and particularly under cyclic, loading with full deformation reversals. Barbell cross-section can resist lateral loading inducing higher v than the rectangular one, and still results in significantly better overall behavior (see Figs. 3 and 4), emphasizing the importance of providing well-confined edge members.

6. Axial Load, N. The value of N significantly affects the stiffness, strength, ductility and energy dissipation capacity of the walls, the presence of moderate compression being highly beneficial.

7. Construction Joints. Concrete and complete concrete and steel construction joints performed satisfactorily, although lap splicing of the reinforcement did not satisfy minimum code requirements and it was located in critical regions which were subjected to $v_{max} = 1.08\sqrt{f'_c}$ (MPa).

8. Flexibility at the Construction Joints. The more flexible the foundation, the closer to it failure of the wall panel begins. However, this change did not affect either the strength or deformation capacity of the isolated wall. Further experimental studies should be conducted to determine how much flexibility is required to induce any significant effects. Recent studies [11] indicate that the flexibility, as well as any movement, of the foundation can have very important effects on the overall response. Rocking of the foundation of each wall of a coupled wall system can increase significantly the inelastic rotation demands on the coupling girders [11].

From the above observations, it becomes clear that, if barbell cross-section walls are proportioned and detailed according to present UBC seismic provisions, they can be very "ductile" and dissipate sufficient energy through stable hysteretic loops to survive the demands of extreme earthquake ground shakings. This holds true even if they are subjected to shear forces above those presently permitted by code, i.e., $V_{max} = \phi 0.83\sqrt{f'_c}$ (MPa) hd, which, considering the specified value of $\phi = 0.85$, gives $V_{max} = 0.71\sqrt{f'_c}$ (MPa) hd. (Values up to $V_{max} = 1.15\sqrt{f'_c}$ (MPa) hd have been resisted.) The amount of displacement ductility that can be used is limited by instability problems. Present code and Paulay's [5] suggested dimensional limitations to avoid instability are not adequate when the required μ_δ under cyclic loading including reversals of deformations exceeds 6 for barbell cross sections and 3 for rectangular cross sections. Dimensional limitations should depend upon the required rotation capacity of the critical region of the wall and type of loading history.

Coupling Girders. The performance of coupling girders during the 1964 Alaska and 1972 Managua earthquakes demonstrated that a conventional approach to designing and detailing these girders results in poor performance. This is not surprising because these girders are often deep in relation to their span.

Thus, significant interaction between shear and flexure--usually disregarded in conventional design procedures--may be present. Furthermore, the deformation capacity, the number of yielding excursions, and the number of plastic rotation reversals demanded from these coupling beams are very large compared with those encountered in beams of ductile moment-resistant frames [7]. Analysis of the experimental data available indicates the following: (1) Compliance with code requirements results in satisfactory hysteretic behavior when $v_{max} \leq 0.25\sqrt{f'_c}(MPa)$; (2) When v_{max} is in the range of 0.25 to $0.5\sqrt{f'_c}(MPa)$, it is necessary to use special web reinforcement. Although the use of intermediate longitudinal bars improves hysteretic behavior, the addition of diagonal reinforcement seems to be more effective in controlling sliding shear at critical regions; (3) The use of conventional reinforced beams where the nominal unit shear stress, v_{max}, can exceed $0.5\sqrt{f'_c}(MPa)$ should be avoided; (4) When $v_{max} > 0.25\sqrt{f'_c}(MPa)$ and particularly when it exceeds $0.5\sqrt{f'_c}(MPa)$ as is usual for coupling beams, the energy dissipation capacity can be improved significantly by placing the main reinforcement diagonally in the beams as has been demonstrated by Paulay [5]. The superior response of diagonally reinforced coupling beams has also been shown in tests carried out by the PCA [12]. Therefore, even in cases of short-deep coupling beams, it is possible to design and construct them so they can offer excellent ductility and hysteretic dissipation of energy.

Concluding Remarks. Ample experimental and analytical evidence indicates it is possible to design and construct very "ductile walls and coupling girders" which could supply frame-coupled wall buildings with stiffness, strength, ductility and energy dissipation capacity in excess of the actual demands, even when these buildings are subjected to recorded or estimated extreme ground motions. This observation is strongly supported by the results of experiments conducted on frame-walls and coupled walls by Paulay [5] and on coupled walls by the PCA [13], and by the observed performance of these types of building structural systems during recent destructive earthquakes. However, there are still several problems requiring further study before specific guidelines and/or reliable code provisions can be recommended for the seismic-resistant design of R/C frame-coupled wall buildings. Some of these are enumerated below.

RESEARCH NEEDS

1. Problems in Estimating Demands. There is a need to develop more reliable methods for estimating the maximum shear that can occur in each story of a complete frame-coupled wall building, of its frame and coupled walls, and of its individual walls. This will require investigation of the (1) effects of foundation movements; (2) variation in coupling girders' flexural and shear stiffness and strength; (3) effects of wall axial forces in the variation of their flexural and shear strength and stiffness (particularly the latter); and (4) interacting effects of frame and coupled walls.

2. Problems in Estimating Supply. For any given or selected wall, there is a need to improve present methods of predicting its shear strength, flexural and shear stiffnesses, particularly in the inelastic range. Conventional methods are inadequate.

3. Problems in Design. Developing optimal methods for the design of coupled wall and frame-coupled wall systems will require investigation into the optimal selection of stiffness and strength of the coupling girders, and the optimal selection of thickness of walls and size of edge members, considering the possibility of using different sizes for the outside and inside edge members.

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