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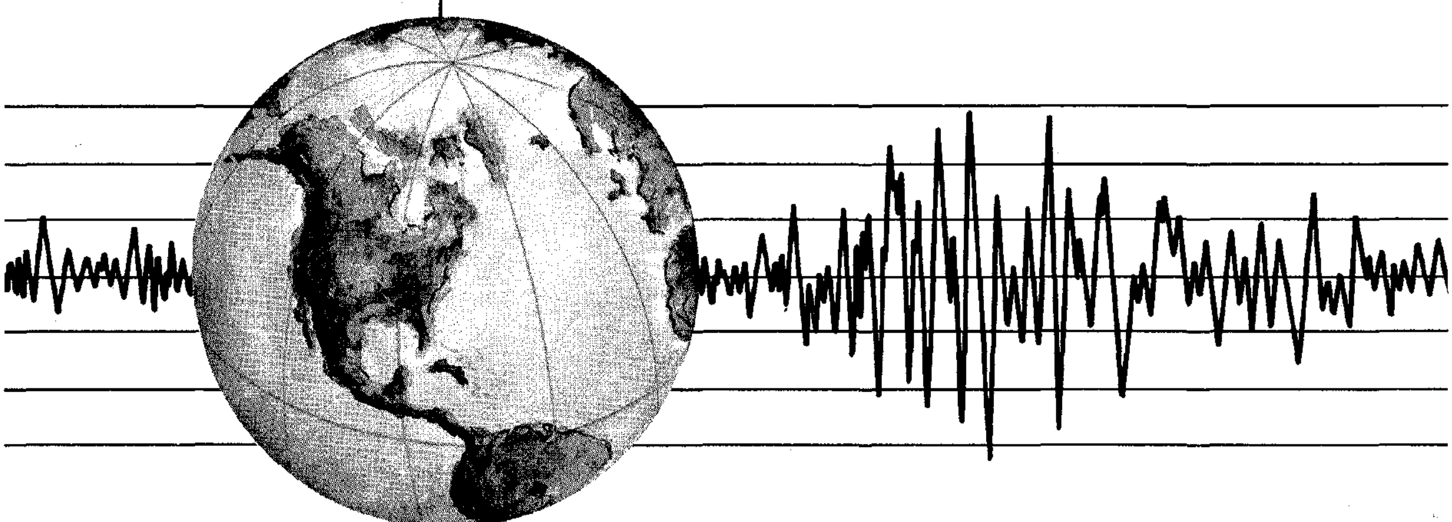
STATES OF THE ART AND PRACTICE IN THE OPTIMUM SEISMIC DESIGN AND ANALYTICAL RESPONSE PREDICTION OF R/C FRAME-WALL STRUCTURES

by

A. E. AKTAN
V. V. BERTERO

Report to National Science Foundation



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The objective of this report is to evaluate the states of the art and practice in seismic design and analysis of R/C frame-coupled wall structural systems. The findings during the course of continuing research at the University of California, Berkeley, for the past decade on "The Seismic Behavior of Structural Components--R/C Frame-Wall Systems," and the relevant literature, constitute the basis for the evaluations reported here. However, the major source of evaluations was based on integrated analytical and experimental investigations of the seismic responses of a 15-story R/C frame-coupled wall structural system, used as the example in these evaluations.

The step-by-step design of the 15-story structure was carried out by considering each pertaining provision of the 1973 UBC, 1979 UBC, and ATC 3-06, which had been considered to represent the state of the practice.

Analytical models of the 1973 UBC designed version of the structure were utilized for elastic and inelastic seismic response analyses, while a 1/3-scale, 4-1/2 story-subassemblage of one of the structure's coupled wall systems was constructed and subjected to numerous experiments. Observations from these studies led to evaluations of the state of the practice in the seismic resistant design and the state of the art in the analytical seismic response prediction of R/C frame-coupled wall structures.

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STATES OF THE ART AND PRACTICE IN THE OPTIMUM SEISMIC DESIGN AND ANALYTICAL RESPONSE PREDICTION OF R/C FRAME-WALL STRUCTURES

by

A. E. Aktan
Associate Research Engineer
Department of Civil Engineering
University of California, Berkeley

V. V. Bertero
Professor of Civil Engineering
University of California, Berkeley

Report to Sponsor:
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College of Engineering
University of California
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July 1982

ABSTRACT

This report is written with the objective of conducting a comprehensive evaluation of the states of the art and practice in seismic design and analysis of R/C frame-coupled wall structural systems. The findings and observations during the course of continuing research at the University of California, Berkeley, for the past decade on "The Seismic Behavior of Structural Components--R/C Frame-Wall Systems," and the existing relevant literature, constitute the basis for the evaluations reported here. However, the major source of evaluations was based on the integrated analytical and experimental investigations of the seismic responses of a 15-story R/C frame-coupled wall structural system, used as the example in these evaluations.

The step-by-step design of the 15-story structure, beginning with the selection of its structural system, to the detailing of the reinforcement of its components, was carried out by considering each pertaining provision of the 1973 UBC, 1979 UBC, and ATC 3-06, which had been considered to represent the state of the practice.

Analytical models of the 1973 UBC designed version of the structure were utilized for elastic and inelastic seismic response analyses, while a 1/3-scale, 4-1/2 story-subassemblage of one of the structure's coupled wall systems was constructed and subjected to numerous experiments. Observations from these studies led to evaluations of the state of the practice in the seismic resistant design and the state of the art in the analytical seismic response prediction of R/C frame-coupled wall structures.

The state of the practice was assessed to lead to a design which had response characteristics considerably different from those depicted by the UBC, and which exhibited collapse limit state response characteristics that may be considered undesirable.

The state of the art of the seismic response prediction of R/C frame-wall structures was assessed to be inadequate in predicting the

axial-flexural and shear capacities of structural components as well as the measured distributions of force and distortion over the structure, at all the limit states of response.

Comprehensive integrated analytical and experimental investigations are needed to advance the states of the art and practice in optimum seismic design and analysis of R/C frame-wall structures.

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Many of the ideas, observations, discussions and conclusions presented in this report were inspired by spirited discussions and exchanges of ideas during weekly meetings between the Principal Investigator, V. Bertero, co-author Dr. A. Aktan, and contributors. These included a large number of graduate students, research staff, and visiting faculty associated with the research projects. The contributions to the investigations reported herein of Professor K. Yoshimura of Oita University, Japan, are especially acknowledged, as are those of Akin Ozselcuk, graduate student. The authors also wish to acknowledge the editing assistance of S. Gardner and figure illustrations by R. Steele.

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1. INTRODUCTION

1.1 General

Structures designed in accordance with the state of the practice are expected to resist "frequent minor earthquakes" without damage, "occasional moderate earthquakes" without structural but with some non-structural damage, and "rare but probable major earthquakes" without collapse [18]. These performance criteria are in some way considered in present code design procedure [21]. The unfactored UBC seismic forces could be related to the "minor earthquake" and represent the service load level. In the case of R/C structures, the factored code design demands could relate to the initiation of the damageability limit state and hence would relate to the concept of the "moderate earthquake." During the design process, the displacements and drifts caused by the service level loads (gravity and wind or minor earthquake) are used to check whether the structure has adequate stiffness for this limit state. The damage level demands are subsequently used to perform the final proportioning of the members.

The displacement and drift limitations at the service and damage levels, imposed by the code in accordance with performance criteria, have made the utilization of structural wall systems desirable in medium-to-tall buildings [8].

Finally, the minimum sizes and amount of reinforcement and particularly special reinforcement detailing required by the code to achieve what is called ductile behavior, is expected to provide the structural system with adequate reserves of energy absorption and dissipation capacity to resist "major earthquakes" without collapse [18]. The designer, however, is not explicitly required to investigate the supply and demand relations of the structure at this ultimate limit state of collapse. Even if this were required it is doubtful that the state of the art is adequate to conduct such a supply and demand investigation in a realistic manner, particularly when the structural system incorporated structural walls. These doubts have motivated this report.

1.2 Objectives and Scope

The main objectives of this report are to assess: (1) Whether the design of R/C frame-wall structural systems, in conformance with the 1973 UBC, 1979 UBC, or ATC 3-06 provisions (which are assumed to represent the state of the practice), would inherently possess desirable ultimate limit state responses; and, (2) Whether the state of the art is adequate to carry out a realistic assessment of the supply vs. demand relations of a R/C structure at all the limit states, particularly at collapse when subjected to the probable extreme earthquake ground motion.

To attain these objectives, analyses are conducted of the results obtained in an integrated analytical and experimental research program which is still under progress, and which has the ultimate objective of formulating optimum seismic design provisions for R/C frame-wall structural systems. A 15-story building with a frame-coupled wall structural system (Fig. 1) was selected as one of the subjects of the analytical and experimental research [2]. This building was used to exemplify the demands and design provisions prescribed by the 1973 UBC, 1979 UBC, and the recommendations of ATC 3-06. The 1973 UBC designed version was observed to possess relatively more favorable ultimate limit state response characteristics and was, therefore, selected for the assessment of the states of practice and art.

2. SEISMIC RESISTANT DESIGN OF R/C FRAME-WALL SYSTEM: STATES OF THE PRACTICE AND ART

2.1 Selection and Initial Proportioning of the Structural System

The selection of the structural system is considered to be the most consequential step in the design process. The main problems associated with this phase are discussed below.

2.1.1 The Types of Wall Systems To Be Used and the Amount and Distribution (in Plan and Elevation) of the Walls

Code provisions do not provide assistance to the designer in decisions concerning the selection of either isolated (single), pierced, or coupled wall systems. These might have considerably different response characteristics and, therefore, require proper identification.

The need for a classification of different wall systems is also observed for the state of the art. Comparisons were made of previous researchers' published experimental investigations of typical coupled wall system conceptions (see Fig. 2), and considerable differences in the overall aspect ratios, member proportions and, therefore, response characteristics, were observed [2,5,6,13,16]. This comparison points out a need to develop a system identification for wall systems before specific and rational design provisions for walls can be established. For the optimum selection of the type of wall system, significant advantages of properly designed coupled wall systems over others were discussed in the literature [8]. Guidelines should be developed regarding the optimum relative span of the coupling girders based on the height of the system, as well as other parameters which will be subsequently discussed.

Guidelines exist predicated on previous earthquake induced damage studies regarding the distribution of walls in plan and in the elevation of the structure. The literature has ample reference to the detrimental results of terminating structural walls before the foundation level, inducing a soft story. Many examples of extensive earthquake damage of R/C frame-wall structural systems were associated with significant discontinuities in the stiffness or mass along the elevation of the structure or eccentric layouts of the walls within the plan [2]. The UBC provisions, however, do not effectively safeguard against the selection of such systems. The Japanese Building Standard Law [10] contains provisions which increase the required ultimate shear strength of stories exhibiting stiffness discontinuities.

Regarding the amount of walls to be used in a building, an explicit minimum requirement does not exist in UBC. ATC 3-06 provisions recommend a minimum of four planes incorporating walls in each direction. Of course, the code drift restrictions at service level may be considered to provide a guide regarding the minimum amount of wall in a certain direction. However, present UBC provisions regarding the empirical estimation of the fundamental period of frame-wall structures does not incorporate the amount of walls and, therefore, can actually mislead the designer in the selection of the structural system. Furthermore, selection of an optimum structural layout would require the assessment of the response at all the limit states, rather than just the service level. To consider the damageability and collapse limit state responses in design, the code provisions should require the analysis of structural response under the rare but probable extreme ground motion. Unfortunately, the states of the practice and even of the art need improvement in defining the characteristics of this extreme design earthquake, and of the dynamic characteristics of the different types of frame-wall structures in the inelastic range. At present there is no agreement on the best index to measure damage, and for commonly used indeces such as the interstory drift or the tangential interstory drift, no reliable tolerable values have been formulated for different types of structural and nonstructural components.

2.1.2 The Initial Proportioning of the Walls and Their Coupling Elements, Frame Elements and the Diaphragm System

For frame-coupled wall systems there are no code guidelines to aid the designer in the selection of the relative stiffnesses of the walls and coupling elements, and of the diaphragm system for optimum structural response. The UBC and ATC 3-06 provisions require a ductile frame system capable of resisting at least 25 percent of the lateral forces to complement the wall system for structures over 160 ft tall. There is no provision, however, regarding the desirable (optimum) relative stiffness of this frame. Although the decision regarding the stiffness of the frame system relative to the wall system plays an important role at all limit states of response, it is of particular importance in defining the stage of response at which the energy dissipating mechanisms of these frames will be activated. If the frame system is too flexible, its contribution to the energy dissipation of the structure, as well as its effectiveness as a restraint, may become relevant only after significant wall resistance deterioration resulting from extensive damage. However, if the frame system is too stiff, the seismic force (strength) demands from the complete frame-wall structural system may increase to an undesirable magnitude.

The relative stiffness of the coupling elements and the walls affect the response of coupled wall structures significantly [2]. As suggested by previous researchers [7,11], it appears to be advantageous to proportion the coupling elements (in conjunction with architectural constraints) so that the stiffness of the structural system becomes insensitive to any further increase in the stiffness of the coupling elements. For the building in Fig. 1, this approach led to the indicated girder dimensions, and the stiffness of the coupled wall system was obtained from elastic theory as 80 percent of the stiffness of a solid wall without the openings [2]. It should be emphasized, however, that a parameter affecting the response of the structure more significantly than the coupling element stiffness was observed to be the coupling element strength. Code design procedure is based on elastic analysis, which results in flexural design demands in proportion with the relative stiffnesses. This does not necessarily lead to optimum coupling element strengths when these elements are proportioned for maximum stiffness. The designer should have a clear understanding that for any selected stiffness of a structural component, different yield and maximum strengths can be assigned to this component by the proper selection of the grade, amount, and detailing of the reinforcement.

The selected cross sectional shape for the walls was barbell, as shown in Fig. 1, as this shape was observed to be more efficient than rectangular for the same weight of concrete [8]. Therefore, from this point of view, it is convenient to select the minimum acceptable thickness for the wall panel. The thickness of the panel at present is controlled by the code requirements (nominal shear stress $v_u \leq 10 \sqrt{f'_c}$ and instability). Inadequate thickness was observed to lead to premature brittle type of failure due to crushing and splitting of the concrete in the case of high axial and shear stress demands. The thickness selected for the wall in Fig. 1 was based on the minimum requirements of the UBC to guarantee its out-of-plane stability. The relative thickness of the panel to the edge member requires particular consideration as this parameter affects the relative shear capacities of the panel and the edge member. If the panel thickness is inadequate, a shear-compressive failure of the panel may occur at early stages of the ultimate limit state response. To utilize the redundancy provided by the edge members during the ultimate limit state, these members should be capable of resisting the shear that would be released after a panel compressive-shear failure. This is recognized by the current Japanese AIJ standards [1] which require an assessment of the design based on the damage level at the failure limit state as well.

The selection of the panel and edge member dimensions for optimum response is a problem which requires urgent investigation. To properly select these variables for optimum ultimate limit state response, development of methods for accurate computation of the state of stress at the critical regions of the panel, as well as establishment of a realistic failure criterion for reinforced concrete under these possible states of stress, are required.

The proportioning of the diaphragm system is another critical step affecting the response of the structure, as the distribution of seismic demands between different lateral force resisting components (i.e., the frame-wall interactions) are particularly affected by the actual axial, shear, and flexural stiffnesses of the diaphragm system. The code requirement to incorporate a ductile moment resisting frame system for buildings over 160 ft tall, and the dimensional requirements for the flexural members of such frames, guide the designer toward the selection of a beam-slab system rather than a flat plate, as the minimum strength requirements of the frame may not be easily supplied by the portion of flat slab meeting the maximum width-to-height ratio permitted for flexural members by the UBC (Sect. 2626(e)1).

A particularly important aspect of the diaphragm system regards the interface of the diaphragm and the wall. The AIJ provisions [1] require a frame to bound the wall panel, guiding the designer to incorporate a beam along the wall-diaphragm interface. This was observed to improve the stiffness of the diaphragm system and to provide an effective restraint to the edge columns of the wall at the ultimate limit state in the case of a panel failure, i.e., the beams do not permit propagation of panel failure from one story to the adjacent one, allowing the beam and column to work as moment resisting frame. Such beams, however, are not required by code and are usually neglected in U.S. practice because of the complexity they cause in formwork.

2.2 Analysis and Design

2.2.1 General

The last decade has witnessed many advances in seismic response analysis of analytical models of buildings, but similar advances have not been achieved in seismic resistant design. Sophisticated computer codes have been developed to carry out linear and nonlinear dynamic analyses of three-dimensional analytical models of buildings. The preliminary design of a building, however, should be available before these analyses are conducted. Although an optimal nonlinear seismic design procedure has been formulated for R/C ductile moment resistant frame [9,23], to the best of the author's knowledge, no similar procedure has been developed for preliminary design of frame-wall structures. Present rigorous linear and nonlinear dynamic analysis procedures cannot be directly applied for preliminary design. Code equivalent lateral force procedures, computations of which are based on an estimation of the fundamental period of the structure through simple empirical expressions, are suited for preliminary design. The reliability of present code expressions to: (1) Estimate the fundamental period of frame-wall structural systems; (2) Estimate the level of the seismic force; and, (3) Obtain the distribution of the seismic force, are questionable. The designer should, therefore, conduct analyses of the preliminary design of the structure using more reliable methods for estimating its supplies and actual demands.

2.2.2 Lateral Force Requirements and Analysis

After a preliminary design is available, it is necessary to establish the gravity and seismic demands. First the provisions regarding the design lateral forces are reviewed. The total seismic force required for the short direction of the building in Fig. 1 was 3.84, 4.96, and 5.47 percent of the building weight, according to the 1973 UBC, 1979 UBC, and ATC 3-06. Incorporating the shears arising from a minimum torsional eccentricity required by these provisions, the seismic forces became 4.53, 5.85, and 5.99 percent of the building weight, respectively. It should be noted that while UBC defines the loads at service level, the ATC 3-06 defines the seismic forces at the first significant yielding of the structure. These forces were computed assuming the zone of highest seismicity and neglecting soil amplification. The expression to estimate the building period was the same for all the codes, resulting in 1.15 sec. A modal analysis of the preliminary design, neglecting any effect of non-structural elements, indicated the fundamental period to be 0.99 sec.

The lateral force demands of 1973 UBC and 1979 UBC are higher for wall systems than frame systems with the same period. For structures over 160 ft tall, if walls are used, it is compulsory to utilize a dual system with a force factor of 0.80. The force factor for a comparable frame system is 0.67, which is 20 percent less. The ATC 3-06, however, require response modification factors of 8 and 7 for the dual and frame systems respectively, as a result of which the dual system is designed for 14 percent less force.

The provisions for the distribution of the lateral forces led to the base overturning moment to shear rations of 0.68 H, 0.71 H, and 0.72 H for 1973 UBC, 1979 UBC, and ATC 3-06, respectively (Fig. 3). The code provisions for the distribution of lateral force are the same for all types of structural systems, despite the considerable differences in the observed mode shapes and displacement patterns of frames and different types of wall-frame systems [8]. This is particularly consequential in the ultimate limit state response of wall systems, as both axial-flexural and shear designs are based on the same lateral force distribution.

In the design of frame systems, the shear design of their components is based on their flexural capacities, while only the axial-flexural design is based on the lateral forces. For walls, both axial-flexural and shear design are based on the code lateral forces. The consequences of this inconsistency will be discussed subsequently.

Linear analysis of the frame-coupled wall system with the specified lateral forces indicated that the maximum interstory drift indexes obtained for the ATC 3-06, 1979 UBC and the 1973 UBC lateral force provisions were 0.065, 0.058, and 0.046 percent of the story height, respectively. ATC 3-06 requires this drift, modified in accordance with the displacement modification factor of 6.5 for dual systems to be less than 1 percent of the story height. 1979 UBC and 1973 UBC require a modification of 1.25 for dual systems and specify the resulting drift to be less than 0.5 percent of the story height. The analyses results indicate that the structure possessed 2.37, 6.90, and 8.69 times larger stiffness than required by the ATC 3-06, 1979 UBC and 1973 UBC provisions, respectively. The substantial stiffness provided by the coupled wall system, several times superceeding the regulations, is the basic reason that these systems are considered effective in providing damage control during minor and moderate earthquakes.

The ATC 3-06 requires the walls and frames to resist the total lateral forces in accordance with their relative rigidities. UBC provisions also require that the walls be capable of resisting the total lateral forces acting independently of the frames. Elastic analyses of the complete structure, as well as only the coupled wall system with the prescribed seismic loading, indicated that the individual demands from the wall system were larger for the latter case, while the ratio of the overturning moment to shear at the base of the wall system was smaller when the total structure was considered. The UBC provision requiring the walls to resist all the lateral force, therefore, led to higher axial-flexural strength requirement.

The distribution of the maximum seismic force demands from one coupled wall system, as a result of the required analysis procedure and modified to incorporate the shears arising from the minimum 5 percent torsional eccentricity requirements of the UBC and ATC 3-06, are shown in Fig. 3. The total seismic force requirement is maximum for 1979 UBC, followed by ATC 3-06 and 1973 UBC, as 5.85, 5.02, and 4.53 percent of one-half the total building weight, respectively. The base overturning moment-to-shear ratio of these seismic force requirements were 0.71 H, 0.60 H, and 0.68 H, respectively, indicating the ratio obtained for the ATC 3-06 demands being the smallest, as a result of considering the interaction between the frames and coupled walls.

2.2.3 Design Requirements

2.3.3.1 Axial-Flexural Design of Walls

The axial-flexural and shear design of walls are carried out using different load and capacity reduction factors. The load factor required for the axial-flexural design of a wall section with respect to the 1973 or 1979 UBC provision is 1.4 for the seismic force, while ATC 3-06 prescribes a factor of unity. Both UBC and ATC 3-06 provisions require specially confined boundary members for walls of dual systems, required to be capable of resisting all axial-flexural effects. These boundary members are designed as axial load members, for which strength reduction factors of 0.7 (0.75 for spiral) in compression and 0.9 for tension are specified. This procedure results in a wall cross section which has a considerably larger axial-flexural capacity than required as the strength provided by the total wall cross section, incorporating the wall steel is underestimated. In the design of interior boundary members of the coupled walls, the demands are computed to be substantially smaller than those for the exterior boundary members, according to the code procedure in establishing these demands. In the ultimate limit state response, however, upon either the loss of the resistance of the coupling girders, or a panel failure, the axial-flexural and shear demands from the interior boundary members would be approximately equal (because of the direct effects of the coupling girders, the shear demands can be larger) to the corresponding demands from the exterior boundary members. It is advisable, therefore, to detail the interior boundary members similar to the exterior boundary members. The distribution of seismic shears required for the factored shear and axial-flexural design demands for the coupled wall, as required by UBC and ATC 3-06 provisions, are shown in Fig. 4.

2.2.3.2 Shear Design of Walls

Because the shear stresses developed in the walls are usually very important, the shear design provisions of the 1973 UBC, 1979 UBC, and ATC 3-06 require particular attention. Since the same seismic force distribution is used to establish both the axial-flexural and shear strength demands, the load capacity reduction factors used for shear design should be expected to provide a higher degree of safety against shear failure as compared to flexural yield. The shear failure of the walls should be avoided or postponed until sufficient flexural yielding takes place to provide the system with adequate energy dissipation, in order to attain the code performance criterion for ultimate limit state response.

The 1973 UBC provisions try to guard against shear failure by using a load factor of 2.8 to establish the shear strength demand as compared to 1.4 for axial-flexural design. The load factor for shear design was decreased to 2 in 1976 and 1979 UBCs. The same load factor, i.e., 1.0, is required for both axial-flexural and shear designs by ATC 3-06. However, a relatively lower capacity reduction factor of 0.6, instead of 0.85 as used by UBC, is prescribed in ATC 3-06 for shear design when shear strength governs the response of a member. Consequently, the shear strength demands along the coupled wall system, including the capacity reduction factors, were obtained for 1973 UBC, 1979 UBC, and ATC 3-06, as shown and compared with the axial-flexural design demands in Fig. 4. The demands from each wall of the coupled wall system are identical as a result of the elastic analysis specified by the codes and the usual assumption of using constant stiffness. The demands for the coupled wall system at the base are 14.92, 13.76, and 8.37 percent of one-half of the building weight for the 1973 UBC, 1979 UBC, and ATC 3-06 provisions, respectively.

In assessing the expected ultimate limit state response characteristics of different walls, designed by different sets of shear demands for axial-flexural and shear strength design, the criterion considered was the ratio of the required shear strength demand at the base of the wall system. These ratios are evaluated in Table 1, indicating that the 1973 UBC provisions demand the largest shear strength as well as the highest ratio of shear strength to the base overturning moment strength.

The base overturning moment and shear strength demands, individually, do not reflect the actual capacities that would be attained by the wall system. Due to a large number of factors, which will be discussed subsequently, the actual axial-flexural capacities of the coupling girders and walls would be significantly higher than the demands required by the code, leading to a substantial increase in the base overturning capacity. The actual shear capacity provided by the wall may be either larger or smaller than the demands required by the code, depending on the actual shear resisting mechanisms of the wall system. The code computations for shear strength do not incorporate the actual, experimentally observed mechanisms of shear resistance for walls, but are based on the observed resistance mechanisms of beams which may lead to misleading design computations for shear, even for the damageability limit state.

The code limits the maximum nominal shear stress to 10 $\sqrt{f'_c}$ to safeguard against shear failure. Even this limitation is not adequate against shear failure as the actual distribution of shear in the wall elements may be considerably different at the ultimate limit state than that depicted from an elastic analysis. The ATC 3-06 uses a limitation of 8 $\sqrt{f'_c}$ rather than 10 $\sqrt{f'_c}$ to compensate for such errors in the obtained shear distributions of walls. However, an optimum and conceptual design against shear failure, for particularly coupled wall systems, cannot be carried out unless the contributions of the boundary members and the panel of each wall, as affected by their respective axial-flexural states, are properly incorporated. The AIJ provisions [1] at least recognize the contribution of these two elements of the wall and incorporate them in determining the ultimate shear capacity of walls.

Despite the differences between the actual capacities which would correspond to the code demands given in Table 1, the ratio of the shear strength demand to the overturning strength demand was considered to be a realistic measure of the relative safety provided by the different provisions against shear failure. Based on this evaluation, the 1973 UBC designed version of the structure was selected as the subject of the subsequent analytical and experimental investigations to assess the states of the art and practice. The reinforcement detailing of the walls based on the 1973 UBC demands are shown in Fig. 5.

2.2.3.3 Design of the Coupling Girders

The state of the practice does not distinguish between the design of the coupling girders of wall systems and the flexural members of frame systems. However, the demands on coupling girders to dissipate energy are usually more severe than those of the girders of a ductile frame. The coupling girders should yield first during the inelastic response of the dual system, and dissipate energy as the initial line of defense, while the structure, through its walls and ductile frames, is expected to retain a substantial portion of its initial stiffness. Because of the deformation mechanism of the walls, and the fact that the coupling girders usually are the first to start yielding, the rotation capacity required from these coupling elements is usually higher than that of the ductile moment resisting frame element. Furthermore, because this element usually has a shorter span than frame girders, the shear stresses are higher. Furthermore, the coupling girders are subjected to a larger number of distortion reversals, particularly at the upper parts of the structure. This is due to the effect of higher modes of response [14] which were more dominant in the generated analytical responses of coupled wall systems as compared to frame systems.

The code design procedure requires the flexural design of the coupling elements based on the demands obtained from elastic analysis. Since these members are generally short, and are proportioned particularly to be stiff, elastic analysis naturally leads to high flexural demands. For the building in Fig. 1, these girders were designed in three groups, as indicated in Fig. 1. The reinforcement detailing of the girders designed with respect to the 1973 UBC demands are shown in Fig. 6. The hypothetical #12 reinforcing bars observed in this figure were selected because they reduce to a #4 bar at the 1/3-scale, which was the scale selected for studying experimentally the behavior of subassemblages of the coupled wall. The slab reinforcement was not incorporated in either the flexural or shear designs, and the width of slab considered in the analysis and design was in conformance with the code provisions. The lateral reinforcement provided to the coupling girders to effectively confine the concrete and restrain the longitudinal reinforcement against buckling, was 2.7 times the amount required by shear, according to the code provisions. The shear stress computed for girder type III was 6.5 $\sqrt{f'_c}$ for a nominal specified value of 4000 psi for f'_c. This shear stress, in conjunction with the maximum tie spacing of 12 in. permitted by the code, was not assessed to lead to adequate hysteretic response of the girder, based on past experimental observations. Therefore, the spacing of the lateral reinforcement was decreased and the amount of ties increased to effectively confine the concrete and restrain the longitudinal bars against buckling. Consequently, the lateral reinforcement actually provided was substantially larger than the amount demanded by the code.

Previous analytical and experimental investigations on the response of coupled wall systems [2] have concluded that the flexural capacity of the coupling girders constituted a critical parameter in design. It was suggested to design these girders so that they provide at least two-thirds of the system overturning capacity through the coupling forces they produced [16]. However, this criterion was proposed after investigations of coupled wall systems with a particular topology, closer to a perforated wall than the coupled wall system conceived in this study, as depicted in Fig. 2. The desirable coupling girder strength for favorable ultimate limit state response should depend on the maximum axial tension and compression forces that can be considered as optimal for the walls or can be tolerated without impairing their strength and energy dissipation capacity. Avoiding net tension or limiting the level of compression so as not to exceed a certain percentage of the balanced point level of the N-M ultimate strength diagram at the base of the walls, depending on the evaluated force-deformation characteristics of that particular wall, were observed to be criteria which would result in a more favorable response at the ultimate limit state, as verified by the analytical and experimental studies reported in this paper.

2.2.3.4 Design of the Frames

The 1973 UBC design of the frames was carried out for the larger of the demands arising from either (1) 25 percent of the total lateral force, or (2) as computed by analysis considering the frames and walls resisting the total required force according to their relative stiffness. ATC 3-06 requires the design to be carried out with respect to the latter criterion only. The assumption on the relative stiffness of the frame and wall members was observed to be important in checking criterion (2). The analysis for this criterion was carried out for cracked transformed wall and coupling girder stiffnesses and uncracked stiffness of the frame components as the wall system may be expected to crack prior to the frame system during low level response. This criterion (2) was observed to yield more critical demands for the top 11 floors of the structure as opposed to criterion (1). The reinforcement detailing for the typical frame members obtained in accordance with the 1973 UBC provisions are shown in Fig. 7.

3. ASSESSMENT OF THE STATES OF PRACTICE AND ART

The assessment of the state of the practice in the design of R/C frame-wall systems was carried out using the 1973 UBC design of the example structure, as this was concluded to possess more favorable ultimate limit state response characteristics as compared to the 1979 UBC and ATC 3-06 designs. Nevertheless, the 1973 UBC provisions have short-comings, some of which were already indicated in the discussion of the design process. In fact, detailing the interior edge members similar to the exterior edge members and increasing the lateral reinforcement of the coupling girders substantially, were modifications to the design provisions to improve the expected behavior. Whether the resulting design, representing the state of the practice, possessed adequate reserves of strength and energy to resist a major earthquake without collapse, was assessed by the following analytical and experimental studies.

3.1 Analytical Studies

The primary objectives of the analytical studies conducted on the two analytical models of the structure shown in Fig. 8, were to generate force and distortion demands of the structure when subjected to intense base motions characterizing the code concept of the "major earthquake," and then to compare these to the available supplies in order to analytically assess the adequacy of present code design procedures and requirements. The adequacy of the state of the art in the analytical generation of the demands and supplies was subsequently assessed through experimental investigations. The main results obtained in the different analyses are presented and discussed briefly in subsequent sections.

3.1.1 Linear Dynamic Analyses

Linear spectral analyses were carried out to generate demands caused by the El Centro and Derived Pacoima Dam accelerograms which are shown in Fig. 9. The acceleration response spectra of these records for $\xi = 5\%$

are compared in Fig. 10. Although the peak accelerations of the accelerograms indicate only 15 percent difference, the acceleration response spectra indicates significantly higher demands of the Pacoima motion at periods around 0.4 sec and for periods higher than 0.8 sec. The fundamental periods of the building, considering the frame-coupled wall systems and the coupled walls alone, were obtained as 0.99 and 1.20 sec, respectively. The periods corresponding to the second and third modes were 0.28 and 0.13 sec for the frame-coupled wall system and 0.34 and 0.15 sec for the coupled wall system, respectively.

The demands at the base of the walls obtained through spectral analyses of the frame coupled wall system using the first three modes of the models are compared to the design demands in Table 2. The general purpose linear analysis code TABS [22] was used for the analyses. Table 2 indicates that El Centro and Derived Pacoima Dam demands for linear response are approximately 2.5 and 5 times the code factored design demands. These ground motions, particularly the latter, were considered, therefore, as representative of the code concept of a "major earthquake" and the structure was expected to exhibit extensive levels of inelasticity in responding to these ground motions, characterizing the ultimate limit state response.

3.1.2 Supplied Strength

An investigation of the axial force-moment-curvature relationships of the representative cross sections of the structure was carried out using the computer code RCCOLA [14]. The objectives were to generate required data on the actual yield strength, ultimate capacity, initial stiffness and deformation hardening characteristics of the typical cross sections. This data was required to prepare inputs for inelastic analyses. The stress-strain characteristics for the reinforcing steel, and unconfined and confined concrete were selected as shown in Fig. 11, based on the assumed nominal strengths and general response characteristics observed in previous studies on material stress-strain characteristics.

The axial force-bending moment interaction relations for the wall cross section, shown in Fig. 12, were derived considering different criteria (based on different limiting strain/stress) for the attainment of the cross section capacity, indicating substantial axial-flexural overstrength.

The moment-curvature responses for type III coupling girder are compared to the factored code design demands in Fig. 13. Zero axial force and no contribution of the slab reinforcement in the assumed flange width of a quarter of the span was considered in the cross sectional analysis. Nevertheless, significant increases in capacity over the code demands are observed.

The above observed overstrengths are due to providing slightly more steel than required, and especially due to the strain hardening of the reinforcing steel and large deformation capacity of the confined concrete of the edge members for the walls.

3.1.3 Collapse State Analysis

A preliminary assessment of the actual overturning capacity and collapse state response of the coupled wall was carried out, assuming the mechanism shown in Fig. 14. The axial forces at the base of the walls at the collapse state, arising from the gravity loads and coupling forces (shear forces of coupling girders corresponding to their flexural capacities), are indicated in Fig. 12. The levels of the compressive and tensile forces are observed to be 1.58 and 8.39 times higher, respectively, than the code design demands. Generated moment-curvature responses of the wall cross section at these axial force levels indicated: (1) The curvature capacity of the wall under such a high level of compression was limited to only approximately twice the curvature at the yielding of the cross section; and (2) The flexural rigidity of this wall under compression was approximately 50 times that of the wall under tension [2]. Consequently, the wall under compression would be expected to attract practically all the shear at the base.

The total base shear force of the coupled wall system at the collapse state would depend on the distribution of the lateral seismic forces, which may be replaced by the resultant acting at its respective location, defined in Fig. 14 by αH. Assuming a base overturning moment to shear ratio of 0.68 H, in accordance with the seismic force distribution defined by the code, the total lateral force at collapse state was calculated as 19 percent of one-half the weight of the building. If all this shear is attracted to the wall under compression at the base, the nominal shear stress of this wall would be 19 $\sqrt{f'_c}$, assuming a nominal concrete strength of 4000 psi. It may be concluded that the depicted collapse mechanism is likely to be preceded by a shear failure of the wall under compression, as the level of shear stress significantly exceeds previously observed capacities in experimental studies [2].

In reality, the frames would also contribute to the lateral resistance, and the base overturning to shear ratio may be different from 0.68 H at the collapse state. Furthermore, the assumed collapse state may not be realized during dynamic response, and the axial forces and shears at the base of the walls may be different than assumed here. Nevertheless, the collapse state analysis serves to indicate an undesirable state of axial force at the base of the wall, arising from the capacities of the coupling girders.

3.1.4 Inelastic Time-History Analyses

The analytical models of both the coupled wall and the frame-coupled wall systems, topological features of which were illustrated in Fig. 8, were subjected to the El Centro and Derived Pacoima Dam excitations shown in Fig. 9, to obtain their inelastic responses. The general purpose inelastic plane frame analysis code DRAIN-2D [12] was utilized for these analyses. The objectives were: (1) To investigate the state of the art in the analytical modeling of R/C structures for time-history analysis; (2) To analytically generate time-histories, maxima and distributions of ultimate limit state force and distortion demands from the structural components; (3) To investigate the contribution of the frame-wall interaction on the system responses; and (4) To generate information to assist in the conduct of integrated experimental studies. The analytically generated demands were subsequently used to assess the design of the structure, based on the provisions of the 1973 UBC, which is assumed to reflect the state of the practice.

3.1.4.1 Analytical Modeling

Analytical modeling of R/C structures for the purposes of inelastic time-history earthquake response analysis requires input from the areas of structural dynamics, finite elements, theory of plasticity, matrix methods, numerical methods (computational techniques and programming), and especially, from the behavior of reinforced concrete. Inelastic time-history analyses of complex R/C systems are presently carried out as tools in the design review process as well as for research, using a considerable number of general purpose computer codes developed for this purpose [17]. The analytical modeling of frame-coupled wall structures poses special problems regarding the reinforced concrete aspect of modeling [3,4]. The topological features of the analytical models of the coupled wall and the frame-coupled wall systems considered in this study were given in Fig. 8, showing that only the planar response of the main structural elements in the transverse direction of the building were considered. The idealizations required to construct the analytical models in conjunction with the computer code DRAIN-2D [12] are discussed in detail in Refs. [3] and [4] and will be briefly reviewed herein.

It is possible to construct different analytical models of the same structure in conjunction with the same computer code as well as constructing more sophisticated and complex models using the recent more capable computer codes [15]. A careful assessment of the state of the art in modeling revealed, however, that the uncertainties and limitations in information regarding the actual hysteretic behavior of reinforced concrete, particularly that of the wall elements, did not justify the utilization of the more sophisticated features available in the topological (finite-elements), plasticity theory and numerical methods aspects incorporated in recent computer codes. Hence, the relatively simple and basic models, shown in Fig. 8, with the main idealizations as briefly discussed in the following, were utilized.

(1) Idealization regarding interactions

(a) Analysis of the planar response necessitated assumptions regarding the actual multi-dimensional natures of the ground excitation and structural response. Interactions between bi-axial flexural effects, changes in the axial force and distortion histories of vertical members, and torsional response, as well as the interactions between the torsional and axial-flexural-shear responses, were phenomena which had to be excluded from the analytical models. These phenomena were observed to require further research and understanding for realistic implementation.

(b) Even in considering only planar responses, two consequential idealizations were required regarding the axial and flexural stiffnesses of the diaphragm system, which were assumed to be infinitely large and small, respectively. These assumptions affected the horizontal and vertical interactions between the different lateral force resisting components. Experimental information regarding the response characteristics of R/C diaphragm systems was observed to be an urgent requirement to improve this aspect of modeling.

(c) Another consequential assumption regarded the interactions between the structural and non-structural components, which were omitted in the models.

(2) Idealizations regarding the foundation and soil

Perfect base fixity was assumed, which was equivalent to assuming infinitely rigid soil and foundation system. This is amongst the most consequential and debatable assumptions, especially considering the mechanisms leading to significant overstrength in overturning resistance of the coupled wall systems, as well as the possible contributions of foundation rocking on the demands from the coupling elements. At present, the uncertainties regarding the actual mechanical characteristics of the soils, particularly their nonlinear behavior under the strain rates imposed by the interacting effects with the building foundation, considerably exceed those regarding reinforced concrete. Analyses with possible bounds on the mechanical characteristics of soil should be conducted to establish guidelines regarding possible effects of the foundation and soil.

(3) Idealization regarding the mass

The total reactive mass of each story was distributed to each of the floor levels above and below the story. The total tributary mass at each floor level was lumped at that floor level. Only the translational inertia forces were considered, neglecting the rotational inertia. Uncertainties regarding the actual reactive rotational mass characteristics of structural systems indicate a need for sensitivity studies on this parameter.

(4) Idealization regarding the discretization and element topologies

One-dimensional line elements were used to represent all the structural elements, including the walls, as shown in Fig. 8. This resulted in limitations in simulating observed deformation patterns and failure modes of the wall members. The location of the wall axis was fixed by lumping both edge members and the panel at the geometric center of the cross-section. This did not enable the neutral axis to fluctuate or to differentiate between the forces and distortions of the two edge members. Using different kinds of 2D finite elements to represent at least the lower floors of the walls, appears desirable. Such elements, however, which should simulate cracking, yielding, and also incorporate the proper failure criteria for the multi-axial stress-state response of concrete, are yet to be developed. The joint regions of the coupling girders in the walls were assumed to be rigid. Although this joint zone was expected to deform, and research to evaluate and incorporate this deformation in the general purpose computer codes for structural analysis is in progress, the common assumption of rigid joint was made.

(5) Idealization regarding the nonlinear response

The geometric effects (beam-column and P-Δ) were checked and observed to be negligible. The material nonlinearities were incorporated at the element level by assuming that the beam and the beam-column elements develop concentrated plastic hinges at each end. These lumped plasticity models enable economically feasible time-history analysis. The simulated local response, and especially the local distortion quantities generated, do not directly correspond to the actual physical counterparts because they neglect the propagation of yield and redistributions within an element and accept only unique values of initial and hardening stiffnesses for the complete element.

(a) Axial-flexural responses of the two-component beam-column elements incorporated an axial-flexural yield envelope. Perfectly elastic response within the envelope, with unique stiffness properties, regardless of the level or sign of axial force was assumed. This was an exceptionally critical assumption for coupled walls since the axial force levels fluctuate significantly in the wall elements and this continuously affects the flexural as well as shear stiffnesses. If the axial force-flexure state indicated yield at one or both ends of the inelastic component of the element, its stiffness was modified for the subsequent time step. Upon withdrawal into the envelope, linear response was resumed. The linear component of the elements simulated the hardening.

The single component beam elements contained rigid-hardening plastic point hinges at each end which incorporated degrading moment-rotation hysteresis characteristics. The main difficulty in preparing inputs for both types of elements was in the synthesis of stiffness, strength, and hardening characteristics derived at the cross section level, into representative characteristics at the element level. This synthesis required assumptions regarding the internal force state (level and distribution) and their histories for all the elements in order to determine the proper average stiffness characteristics.

Incorporation of a tri-linear primary response which includes the cracking of the elements, is observed to be a necessary improvement in defining the structure's initial response characteristics more accurately. This is important since the initial structural periods and mode shapes affect the subsequent inelastic response. Other idealizations regarding axial-flexural responses of the elements included assuming elastic axial force-axial distortion relations as well as neglecting the effects of fluctuating axial force, bond and shear on the flexural hysteresis. It is known that all these effects are of importance in the axial-flexural response and, particularly, in the degradation of flexural hysteresis.

The inputs for the stiffness, strength and hardening characteristics of the critical elements are presented subsequently in assessing supply vs. demand relations.

(b) Shear responses. The shear response of both kinds of elements (beam and beam-column) were incorporated by adding the elastic shear deformation terms in the stiffness, and due to the limitations of the analytical model, the contribution of shear to the element deformations had to be assumed as constant and as represented by the contribution at the elastic stage. This is not a correct representation of observed experimental responses of wall elements, which exhibited a significant reduction in shear stiffness upon flexural yielding of the wall [2]. Reversal of flexural yielding resulted in a sliding of the wall panel, exhibiting a predominantly shear mode of deformation. The level of the reduction in shear stiffness depended on the level of shear stress associated with flexural yielding and the level of axial compression (or tension, in the case of coupled walls) in the wall. Considering that the contribution of shear to the overall deformation may exceed the contribution of flexure due to such a sliding mode of deformation, the inadequacy of representing shear deformations by constant and elastic terms becomes obvious. Efforts to check the level of shear in the element and restrict this to the shear at flexural yielding, by introducing additional hinges at the ends of the elements [19], fail to incorporate the actual sliding shear mode of deformation which should be simulated in order to realistically represent the interactions between the walls and the frames. Unless the actual shear response of wall elements are simulated by using more complex models than a line element for the wall, the analytically generated post-yield responses of frame-wall structures, especially when wall response governs the structural response and shear stress of wall at first flexural yielding is high, may contain significant errors.

(6) Idealizations regarding the numerical aspects of the models

The integration of the equations of motion were carried out assuming constant acceleration in each time step. The element states (stiffness and end forces) were determined subsequently and if found in error, corrective forces were applied in order to avoid the accumulation of such errors, at the next time step [12]. Consequently, equilibrium is not satisfied and significant imbalances may have occurred depending on the time step and characteristics of the motion as well as the nonlinear response. A time step of 0.02 sec was selected based on the predominant periods and trial analyses of short durations using smaller time steps. A number of indications regarding errors in element force-distortion responses, due to the coarseness of the time step, were observed subsequent to the analyses. These were, however, assessed to have affected the global response histories insignificantly.

(7) Other idealizations

A number of other assumptions and idealizations exist that were not considered to be as consequential as those discussed [4]. The assumed 5 percent viscous damping, neglecting the effects of strain rate on the strength of the elements, and the duration of the analyses being limited to the first 10 sec of the accelerograms, were among these.

3.1.4.2 Generated Demands and Supplies

The detailed results of the inelastic analysis were reported elsewhere [4]. Only the most relevant and pertaining results are discussed in the following. The time-histories of lateral displacement, obtained for the El Centro and Pacoima responses of the frame-coupled wall model, are shown in Figs. 15 and 16. The walls remained elastic during the El Centro response, while the coupling girders (except 1st, 14th and 15th floors) and a number of the shorter frame beams (between 6th and 14th floors) developed plastic hinges at each end. The maximum top displacement was 0.23 percent of the building height and the maximum interstory drift was 0.28 percent of the story height, which is less than 0.4 (i.e. 0.5 x k) percent of the story height, the limitation prescribed by UBC and which may be interpreted as the initiation of the damageability limit state.

As indicated in Fig. 16, when the walls were subjected to the Pacoima motion, they developed plastic hinges during the first major displacement excursion. This excursion was induced by the first large acceleration pulse indicated in Fig. 9. The hinge pattern at 3 sec is shown in Fig. 17. The frame columns, which remained elastic, and deformation hardening at the plastic hinges avoided the attainment of a collapse mechanism. The maximum top displacement was 0.72 percent of the building height and the maximum interstory drift was 0.9 percent of the story height. The wall under tension released a significant amount of the shear it was resisting at 3.0 sec, as a result of the second plastic hinge which formed at the base of the second floor. The second floor level had larger tension than the first floor level at this time. The sudden release of shear in the wall under tension is suspected as being unrealistic and is considered to be a consequence of the discussed limitations of the one-dimensional wall members with lumped plasticity. The relevant response quantities and their maximum values obtained during the analyses of the frame-coupled wall model are given in Table 3.

The elongations in the inelastic effective periods, shown in Table 3, are measures of the decrease in the average stiffness of the structure due to inelasticity.

The maximum top displacement and axial-flexural responses of the coupled wall system at the base occurred at the time of the maximum overturning moment, for either ground motion.

The maximum base shear occurred before the maximum overturning moment during the El Centro response and after the maximum overturning moment during the Pacoima response. The distribution of seismic shears along the height of the coupled wall system at the times of maximum base shear and overturning moment for the El Centro and Pacoima analyses are shown in Figs. 18 and 19. The base overturning moment to shear ratios corresponding to the seismic shear distributions indicate that using the same lateral force distribution for axial-flexural and shear designs of the walls, which were shown in Fig. 3, can be misleading in designing against shear.

The contribution of the frames to the responses of the structure was investigated by comparing the responses of the coupled wall and frame-coupled wall models to the ground motions. The frames, as designed, provided approximately 35 percent and 50 percent of the structural stiffness at the 7th and 15th floors, respectively, as evaluated by comparing the lateral displacements of the two models when subjected to the code lateral forces. The frames were observed to decrease the lateral displacements and member disortions by their contribution to the structural stiffness and energy dissipation in the order of 25 percent for the Pacoima responses. The maximum base shears of the coupled wall were increased in the order of 10 percent when the frames were considered. The particularly important contribution of the frames was observed in the Pacoima response, where the redundancy they provided appeared to be consequential in avoiding collapse, since a mechanism state was avoided only because the frame columns remained elastic. A mechanism state was attained during the response of just the coupled wall model to the same ground motion.

3.2 Experimental Studies

A 4-1/2 story, 1/3-scale model subassemblage of one of the coupled wall systems of the building was constructed and tested as shown in Fig. 20 [2]. The choice of the 1/3 scale was to utilize regular concrete and reinforcing steel as well as detailing and construction practices which were in accordance with the state of the practice. This led to a realistic simulation of phenomena including cracking, bond, aggregate interlock, spalling, and splitting of concrete, propagation of yielding and redistributions of force between different components, which were all critical in governing the axial-flexural and shear responses. The selected scale of the model also enabled measurements of local response characteristics such as the variations of the strain components along the wall panels, strains and attenuation of strains on reinforcing steel at critical locations, axial distortions of columns, girders, and the diaphragm, rotations along the girders and walls, and others, which may not have been possible to measure reliably at smaller scales because of limitations of instrumentation

One of the important aspects in instrumentation was the use of internal force transduscers to monitor the internal forces at the midspans of coupling girders. This enabled evaluation of the distribution of the internal forces of the complete subassemblage by statics at any load stage rather than just by estimating the distribution.

The two lateral actuators indicated in Fig. 20 applied equal shear forces to each wall and were coupled to the vertical actuators, within an electro-hydraulic servo control system, programmed to simulate the lateral shears and vertical gravity forces as well as coupling forces and bending moments at the fourth story in order to satisfy the proper force boundary conditions of the subassemblage [2].

The total lateral force-first floor edge displacement envelopes of the two walls, obtained under a lateral force history derived from the generated Pacoima response of the structure, are shown in Fig. 21. The lateral displacements of the wall under compression are observed to be consistently larger, indicating a substantial growth of the diaphragm system. The growth increased with each consecutive limit state. The main limit states, which are indicated by number in Fig. 21, are the following: (1) flexural cracking of tension wall; (2) diagonal cracking of tension wall; (3) diagonal cracking of compression wall; (4) yielding of the coupling girders; (5) yielding of the tension wall; (6) yielding of the compression wall and spalling of the concrete cover of its exterior edge column; (7) crushing of the panel of the compression wall due to a combination of high axial and shear stresses near the exterior edge columns; and, (8) shear failure of the exterior edge column of the compression wall.

It was measured that most of the growth of the diaphragm occurred along the wall in compression (approximately 60 percent), followed by the coupling beam (approximately 30 percent) and the wall under tension (approximately 10 percent). It was subsequently evaluated that the discrepancy between the growths of the walls under compression and tension was approximately equal to the difference in the shear displacements (lateral displacement due to the shear mode of distortion) of the two walls. The beam, although subjected to axial compression, exhibited growth because of flexural hysteresis and the associated accumulation of plastic strains in the reinforcement.

A significant observation from the envelopes in Fig. 21 regards the overstrength of the structure. The actual lateral force capacity was measured as 21.5 percent of the weight of one-half of the building, i.e., more than three times the code factored demand of 6.3 percent of the same weight. The main mechanisms for this overstrength were the increases in the axial-flexural capacities of the coupling girders and the wall under compression. The coupling axial forces arising from the shears of the girders contributed 60 percent of the overturning resistance. The flexural resistance of the walls under compression and tension contributed the remaining 34 and 6 percent, respectively.

The moment-rotation responses of the coupling girders were measured along a 4 in. gage length adjacent to the wall and included the fixed end rotation. After evaluating the average rotations of the girder along the 4 in. gage length, by using the curvatures computed from strain readings in this region and subtracting from the total measured rotation, the fixed end rotations were obtained. The rotations evaluated from the curvatures were observed to be less than 10 percent of the total measured rotation. The moment-rotation responses of the third-floor girder (type III), obtained during a typical half-cycle of loading, and the contribution of the fixed-end distortions to the total measured rotations, are presented in Fig. 22.

The typical redistributions of shear force at the base of the coupled wall system, during a half-cycle of loading, are illustrated in Fig. 23. It is observed that the wall under compression attracts 85 percent of the total base shear even at the service load level defined by 1973 UBC. The differences between flexural rigidities, evaluated from moment-curvature responses of the two walls subjected to the axial force states corresponding to the service load level, were not sufficiently large to explain the extent of the measured redistributions because the redistributions were caused by both flexural and shear stiffnesses of the two walls. The shear stiffnesses were, therefore, verified to have been more affected than the flexural stiffnesses due to the different levels of axial force, which resulted in the measured extent of the redistribution. Nominal shear stresses of 1.6 $\sqrt{f'_c}$ and 16.2 $\sqrt{f'_c}$ were evaluated for the walls under tension and compression, respectively, at the failure of the panel of the wall under compression.

3.3 Comparison of Code, Analytical, and Experimental Responses

3.3.1 Wall Axial-Flexural Strength

The axial-flexural strength interaction curve of the wall cross section, defined as the available supply during analyses, is shown in Fig. 24, and compared with the analytically generated demands as well as code demands. The maximum strength interaction curve, estimated to be attained by the wall under compression during the testing of the specimen, is indicated in the same figure, together with the measured axial force-flexure pair at the failure of the wall which was under compression. The substantial increase in the attained strength with respect to the calculated was attributed to: (1) larger yield strength and maximum capacity of the actual reinforcing steel as opposed to the hypothetical values in Fig. 11; (2) increases in concrete compressive strength and further contributions to the capacity due to confinement; and (3) inadequacies in the analytical computational procedures, such as the inability to simulate the redistributions of stress over the wall cross section. The actually attained flexural strength was approximately 6.0 times larger than the code design demand, and 1.4 times larger than the analytically generated supply. It was not possible to measure the actual capacity of the wall under tension as this wall did not reach even its computed flexural capacity at the shear failure of the wall under compression.

3.3.2 Coupling Girder Flexural Strength

The generated moment-curvature responses of the type III coupling girder is presented in Fig. 13. The typical measured moment-rotation responses of the third floor beam, type III, is presented in Fig. 22. The factored code demand and the analytically derived strength which was used to define the supply in the analyses, were converted to the model scale and are indicated in the same figure. The measured yield strengths in the positive and negative (tension in flange) bending directions are observed to be approximately 1.67 and 1.98 times the analytically computed values and 1.97 and 2.45 times the factored code demands, both respectively. The attained overstrengths over those calculated were caused by larger yield strength and ultimate capacity of steel, compressive axial force in the beam arising from the shear redistributions in walls, the hypothetical underestimated effective flange width as opposed to the actual slab contribution, and the neglection of the contribution of slab steel, which was observed to be effective over the entire width of the slab, representing half of the adjacent transverse span during the ultimate limit state response.

The significantly higher measured hardening stiffness over the analytically generated value and the substantial fixed end rotation observed in the experimental responses were not incorporated adequately in the analytical model. Figure 22 indicates that because of the fixed end rotation, the initial stiffness was substantially overestimated. On the other hand the yielding strength, and particularly the hardening characteristics, were underestimated in the analyses, consequently, the maximum strength was significantly underestimated.

3.3.3 Shear Strength of the Wall

In evaluating the shear strength of the wall, it is important to recognize that according to the analytical prediction, the maximum shear strength demands do not occur simultaneously with the maximum axial-flexural strength. The shear strength demands from one wall, based on code and analyses, are indicated in Fig. 25. The supplied shear strength based on the code provisions is compared in the same figure to the attained shear strength of the walls during the tests. The walls under tension and compression are observed to provide 0.28 and 2.5 times the computed supply. An approximate assessment of whether these measured supplies would have been adequate in the case of the dynamic responses may be carried out by comparing the average measured strength to the analytical demands. It is observed that the average capacity is inadequate to avoid shear failure during the Pacoima excitation. In fact, it may be considered that if the actual axial-flexural strengths of the walls and coupling girders had been incorporated, the analytically predicted El Centro demand may also have exceeded the average supply. The shear failure in dynamic response is more likely to occur if the maximum shear demand occurs after flexural yielding, because of the deterioration of the shear resisting mechanisms of the panel due to extensive cracking, yielding, and spalling. The maximum shear demand in the El Centro response is not associated with flexural yielding since the walls remained linear. Hence, although an explicit assessment regarding the possibility of shear failure is impossible, it appears likely that the structure would survive El Centro.

3.3.4 Structural Force-Displacement Response

The lateral displacement of the fourth floor of the coupled wall system, obtained from linear analysis using cracked section stiffnesses, incorporating elastic shear deformations and under the 1973 UBC loading (service level), was 0.036 percent of the total height at this floor. The corresponding average lateral displacement measured during the experiments was 0.047 percent of the total height at this floor, indicating that the analytical model was 1.31 times stiffer than the test specimen. This was attributed to: (1) The analytical model did not represent the actual extent of the shear deformation of the wall elements even at the service load stage; (2) The analytical model did not incorporate the fixed end rotations at the base of the walls or the uplift and distortions of their foundations; (3) The analytical model overestimated the initial stiffness of the girders by neglecting the fixed end rotations of these girders at the wall interface.

Even the experimentally obtained stiffness of just the coupled wall system, isolated from the frames, however, was adequate to satisfy the story drift code demand for the complete structure. The lateral force-average drift envelope obtained during the test at the top of the specimen, considering the interstory drift at the fourth floor to be representative of the maximum interstory drift of the structure, is compared with the response implicitly assumed by the code, as reflected in the commentary of the SEAOC provisions [18], in Fig. 26. The contrast in the experimental and code conceived responses is striking. The stiffness and strength of the coupled wall is observed to significantly exceed the stiffness required by the code and the strength based on the code provisions. The energy dissipation capacity of the structure, however, is not as large as the capacity implicitely assumed by the code, based on elastic-plastic, stable hysteretic response and a ductility of 5.

4. CONCLUSIONS AND RECOMMENDATIONS

The main specific conclusions reached during the evaluations and discussions of the results presented in this report are given below. Recommendations to improve the present state of the practice and of the art are also offered.

(1) UBC does not provide the designer with a guideline in the selection of the relative stiffness of the frames and the walls other than the requirement that the frame should be designed to resist at least 25 percent of the total base shear.

(2) UBC does not offer guidelines regarding the proper layout of the frame-wall structural system, i.e., avoidance of significant discontinuities in stiffness and mass, minimum number of walls, layout of walls in plan for optimum torsional stiffness and diaphragm behavior of floor slabs.

(3) UBC does not define and differentiate between different types of wall systems, i.e., single or pierced or coupled walls.

(4) UBC does not provide any guidelines regarding the design of the different components of coupled wall systems. Guidelines regarding the relative stiffness and strength of the coupling girders and reinforcement detailing of these girders, as well as the thickness and details of wall panels and whether to have beams bounding the panels, should be developed.

In selecting the strength of the coupling girders for optimum structural response at all the limit states; their axial-flexural capacity should be carefully selected based on tolerable limits of compression or tension force at the critical regions of the coupled walls. Experimental results have indicated that designing these girders to provide 60 percent of the overturning capacity of the coupled wall system led to undesirable levels of axial force in the walls leading to a semi-brittle failure of the wall panel under compression.

(5) The present empirical expression suggested by UBC for the estimation of the period of the frame-wall structures should be revised. Expressions which give possible bounds of the period depending on the amount of walls, and their distribution in plan and height, and on the characteristics of the wall systems, should be incorporated for preliminary design.

(6) Use of linear elastic analysis procedure for the estimation of the flexural demands of the coupling girders did not enable control of the stiffness and flexural capacity fo these elements individually. The designer should be able to prescribe an optimum distribution of relative axial-flexural yielding and ultimate strengths of the structural members independently of the distribution of the relative stiffnesses of these members.

(7) The UBC requirement that "the shear walls acting independently of the ductile moment-resisting portions of the space frame shall resist the total required seismic forces," results in an increase in both the axial-flexural demands and the moment-to-shear ratio at the base of the walls and, therefore, may lead to an unconservative shear design.

(8) The design seismic force distribution along the elevation of the structure prescribed by the UBC are both axial-flexural and shear designs are the same. The distributions obtained during linear or nonlinear dynamic responses at the times of the maximum axial-flexural (overturning) strength demand, and the maximum base shear strength demand, are different such that the base overturning moment-to-shear ratio of the structure, evaluated at these times, differed by as much as 3.7 times, depending on the ground motion and the dynamic characteristics of the structure.

(9) The shear design procedure and the shear strength evaluation procedure that are embodied in UBC, require modification. The shear design should be based on the demands obtained considering the actual axial-flexural capacities of the wall, and should be that which is associated with the lowest possible overturning moment-to-shear ratio at the base, and

assuming a distribution of the shear amongst the coupled wall according to the actual relative stiffness and actual axial-flexural capacity of each wall in the structure. In evaluating the available shear strength, the contributions of the edge members (column and beams), panel concrete, and panel reinforcement should be recognized and incorporated.

(10) The present techniques of estimating the supplies of axial-flexural stiffness and strength of coupling girders and the walls are observed to be inadequate. In evaluating the stiffness of coupling girders, the reduction of the end fixity due to fixed end rotation (i.e., bond slip) should be incorporated. In evaluating the strength of the coupling girders, the possible contribution of the slab steel and axial compression arising from the redistributions of shear between the two walls should also be incorporated. In evaluating the stiffness of the walls, the actual extent of the contribution of shear distortion, the effect of axial force on flexural stiffness, and effects of possible foundation movements should be considered. In design and/or evaluating the strength of the wall, the complete cross section should be considered rather than just considering that the edge columns acting as axially loaded members should resist all the vertical stresses resulting from the different loads acting on the wall.

For evaluation of the strength of both the coupling girders and the walls, the material characteristics require a careful assessment. The actually realized mechanical characteristics of concrete and steel were observed to be significantly different from the nominally prescribed characteristics, in many cases resulting in a significant increase in the axial-flexural strength of the structural member.

In addition to these specific conclusions, two more general conclusions regarding the state of the art and practice in the design and analysis of R/C frame-wall/coupled wall structures may be formulated as follows:

(1) The response characteristics of frame-wall structural systems, designed according to the UBC provisions, may be considerably different from the response conceived by this code. The structure, designed in accordance with the 1973 provisions, was assessed to possess

substantial reserve of axial-flexural strength and adequate energy to withstand the El Centro ground motion with minor damage but failed to survive semi brittle shear failure during the Derived Pacoima ground motion. This assessment was based on the analytically generated demands and experimentally measured supplied. The shear failure was caused by mechanisms of overstrength in the axial-flexural capacities of the members, which resulted in an axial-flexural capacity measured as 3.22 times the corresponding strength demanded by the 1973 UBC.

The shear strength demand and the associated load and capacity reduction factors prescribed by 1973 UBC, as well as the analysis procedures and shear design guidelines of this code, were inadequate against the 3.22 times increase in the axial-flexural capacity of the structure, leading to the semibrittle shear failure of the wall. The 1979 UBC or ATC 3-06 provisions were assessed to lead to even less conservative shear design than 1973 UBC provisions.

(2) The state of the art of the seismic response prediction of R/C frame-wall structures was assessed to be inadequate in predicting the axial-flexural and shear capacities of structural components and the measured distributions of force and distortion over the different elements of the structure at all the limit states of response. This was caused primarily because of the uncertainties in determining the mechanical behavior of R/C and the dynamic response characteristics of the elements and their joints. Many limitations regarding the finite-element and computational aspects of analytical modeling in defining the actual response characteristics and failure criteria were also observed, which contributed to the discrepancies in the predicted and observed responses. Integrated analytical and experimental research conducted on sufficiently complex and large-scaled models of R/C structural systems was observed to be necessary to advance the states of the art and practice in optimum seismic design and response prediction of reinforced concrete frame-wall structures.

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T A B L E S

TABLE 1: REQUIRED OVERTURNING AND SHEAR STRENGTH AT THE BASE OF THE COUPLED WALL SYSTEM

	1973 UCB	1979 UCB	ATC 3-06
TOTAL FACTORED BASE OVERTURNING MOMENT DEMAND (% WH)	4.31	5.81	3.01
TOTAL FACTORED* BASE SHEAR STRENGTH DEMAND (% W)	14.92	13.76	8.37
RATIO OF SHEAR TO OVERTURNING STRENGTH (1/H)	3.46	2.37	2.78

* Including capacity reduction factors

W = Weight of one-half of building

H = Height of building above ground

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TABLE 2: LINEAR ELASTIC DEMANDS AT BASE OF AN INDIVIDUAL WALL OF THE WHOLE FRAME-COUPLED WALL STRUCTURAL SYSTEM

DEMAND / INPUT	SHEAR FORCE (kips)	AXIAL FORCE COMPRESSION (kips)	AXIAL FORCE TENSION (kips)	FLEXURE (10^3 kip-in.)
1973 UBC FACTORED DESIGN DEMANDS	(2.8E): 1084	1.4 (D+L+E): 5544	(0.9D+1.4E): 489	(1.4E): 269
EL CENTRO	2846	12386	7846	1205
DERIVED PACOIMA DAM	5777	23627	19087	2501

TABLE 3

PERIODS AND RESPONSE MAXIMA ATTAINED FOR THE FRAME-COUPLED WALL MODEL

Response Quantity	El Centro Response	Pacoima Response
1) Period based on code	1.15 secs.	1.15 secs.
2) Fundamental period, linear	0.99 secs.	0.99 secs.
3) Effective period, nonlinear	1.52 secs.	1.60 secs.
4) Maximum top displacement index (%H)	0.23 at 5.8 secs.	0.72 at 3.1 secs.
5) Maximum interstory drift index (%h)	0.28 at 5.8 secs., 7th Floor	0.90 at 3.2 secs., 12th Floor
6) Maximum plastic rotation of wall under compression	Remained elastic	0.00124 rads., at 3.04 secs.
7) Maximum plastic rotation of wall under tension	Remained elastic	0.00175 rads., at 3.04 secs.
8) Maximum coupling girder plastic rotation	0.0036 rads. at 5.8 secs., 7th Floor	0.02 rads. at 3.26 secs., 13th Floor
9) Maximum coupled wall system base shear	2675 kips at 4.8 secs.	4053 kips at 3.5 secs.
10) Overturning moment with maximum base shear	2986×10^3 kip-in. at 4.8 secs.	3720×10^3 kip-in. at 3.5 secs.
11) Maximum coupled wall system overturning moment	3870×10^3 kip-in. at 5.8 secs.	5322×10^3 kip-in. at 3.1 secs.
12) Base shear with maximum overturning moment	1522 kips at 5.8 secs.	1506 kips at 3.1 secs.
13) Maximum wall axial compression	5646 kips at 5.8 secs.	6267 kips at 3.1 secs.
14) Maximum wall axial tension	1090 kips at 5.8 secs.	1671 kips at 3.1 secs.
15) Maximum wall base moment	419×10^3 kip-in. at 5.8 secs.	1073×10^3 kip-in. at 3.1 secs.
16) Maximum coupling girder moment	2.69×10^3 kip-in. at 5.8 secs., at 7th Floor	3.13×10^3 kip-in. at 3.26 secs., at 13th Floor
17) Maximum coupling girder shear	299 kips at 5.8 secs., at the 7th Floor	351 kips at 3.26 secs., at 13th Floor

h = Story height, 12 ft.

H = Total height of the structure, 180 ft.

F I G U R E S

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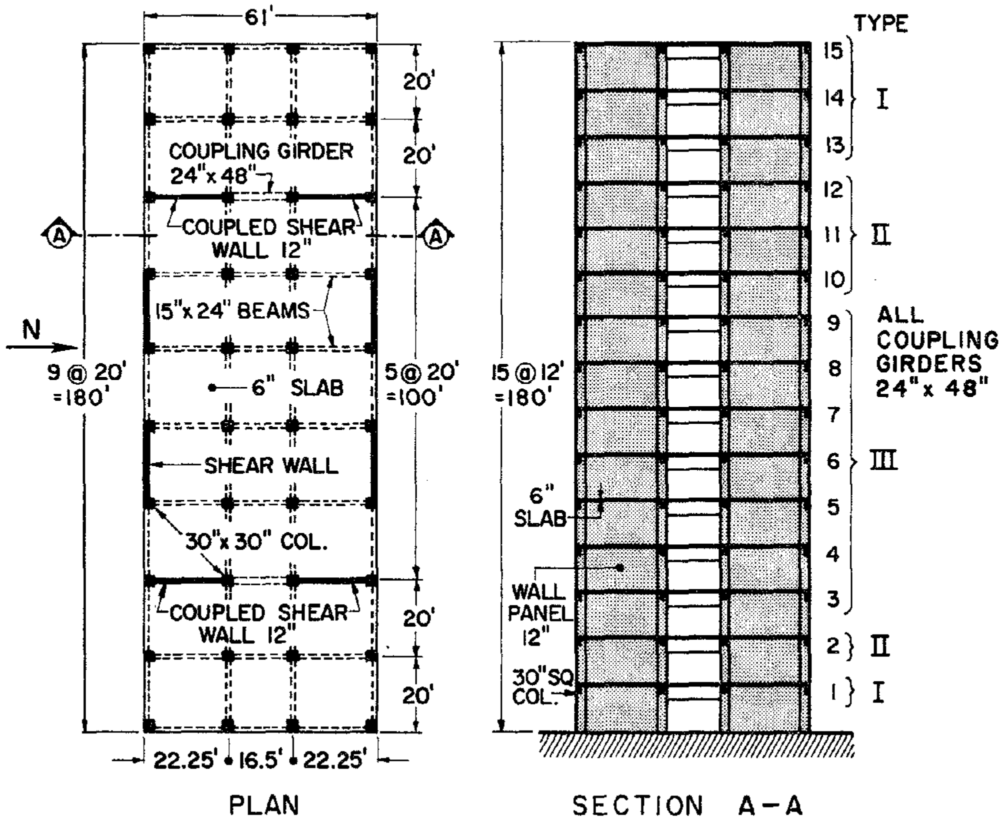
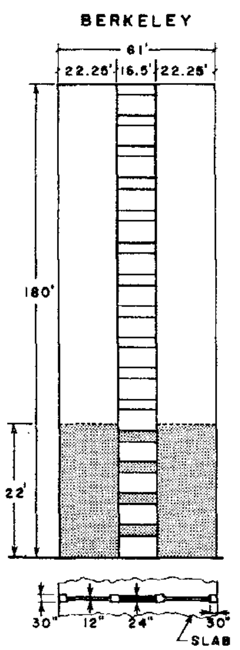


FIG. 1 THE 15-STORY BUILDING STRUCTURAL SYSTEM



	Berkeley	New Zealand	P.C.A.	Urbana	Urbana
Specimens Tested	1	2	1	5	4
No. Stories/Height	15@12'	7@10'	6@9'	6@9'	10@9'
Girder Dimensions (b x d in)	24x48	12x48	12x20 30x24	12x27 12x18	12x18
Scale of Specimen	1/3	1/4	1/3	1/12	1/12

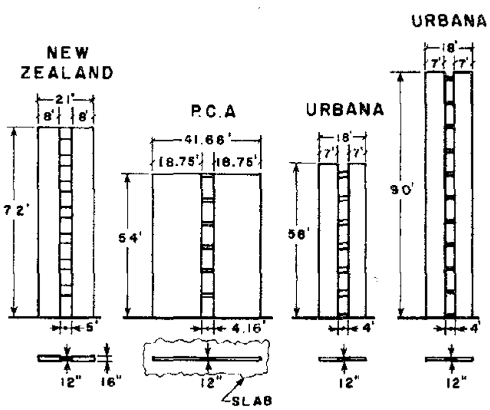


FIG. 2 PROTOTYPES OF PREVIOUSLY EXPERIMENTALLY INVESTIGATED COUPLED WALL SYSTEMS

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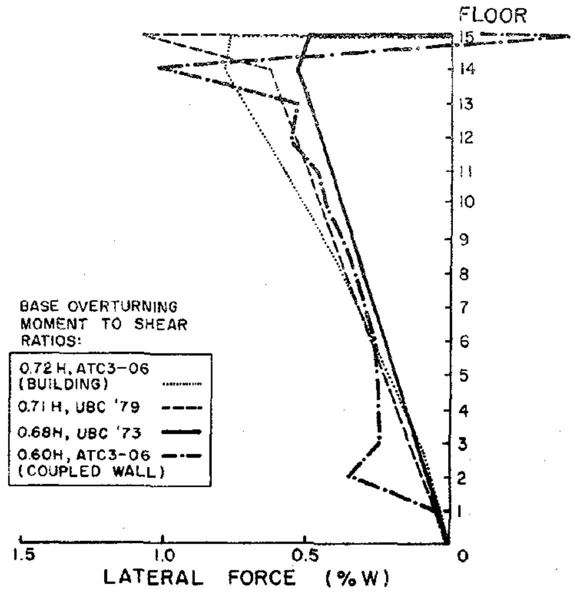


FIG. 3 DISTRIBUTION OF LATERAL FORCES BASED ON DIFFERENT PROVISIONS

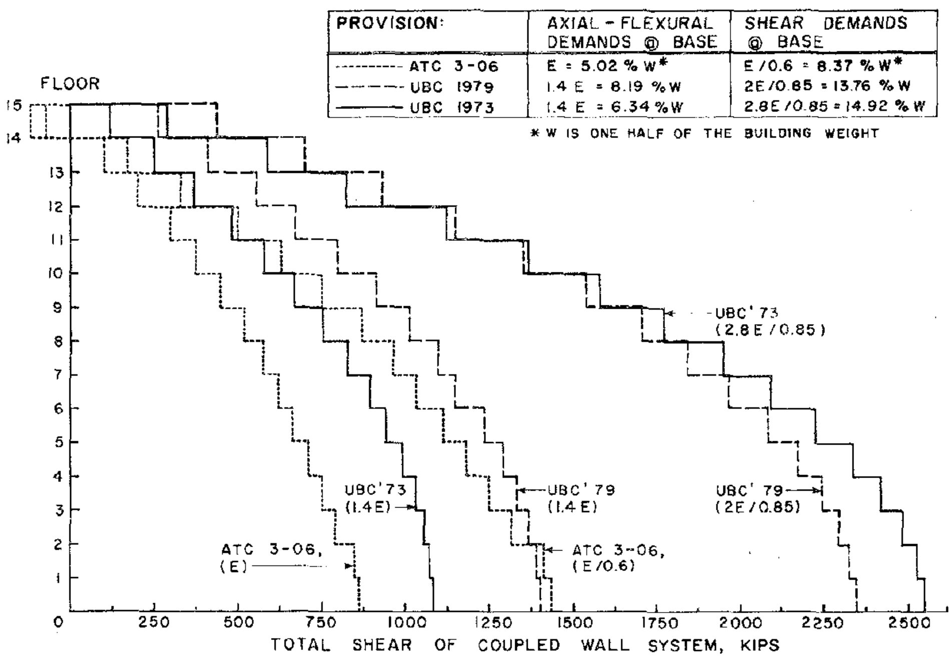


FIG. 4 CODE STORY SHEAR DEMANDS FOR AXIAL-FLEXURAL AND SHEAR DESIGN OF THE COUPLED WALL BASED ON DIFFERENT PROVISIONS

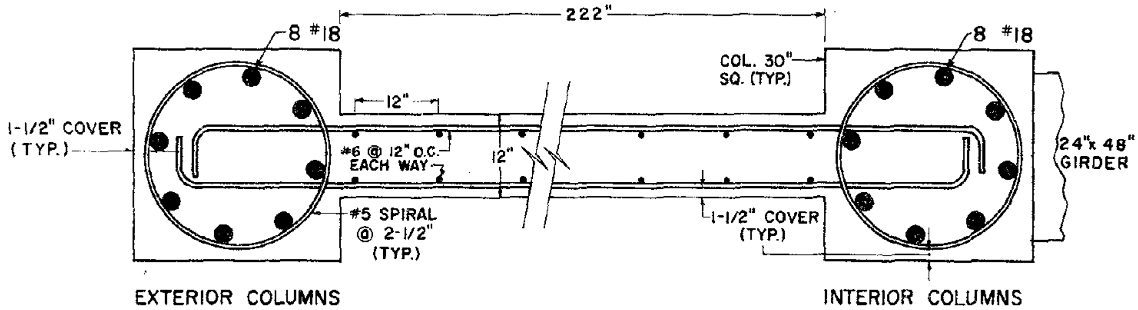


FIG. 5 WALL CROSS SECTION DESIGNED BY UBC-73 PROVISIONS

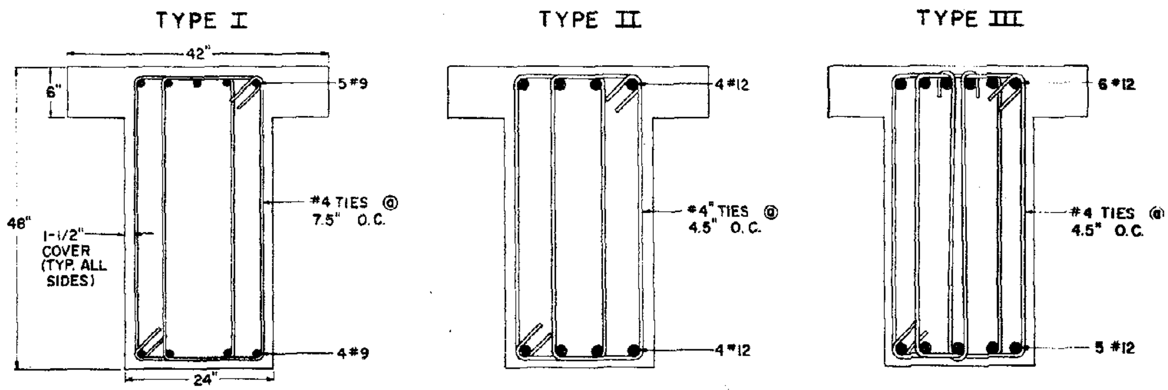
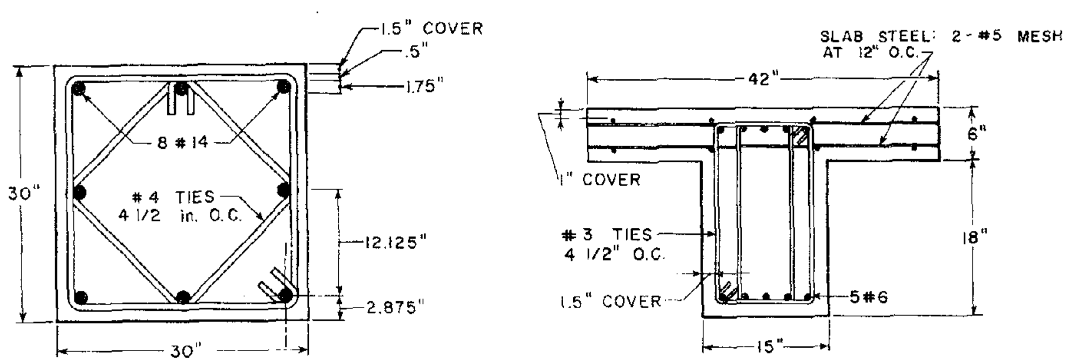
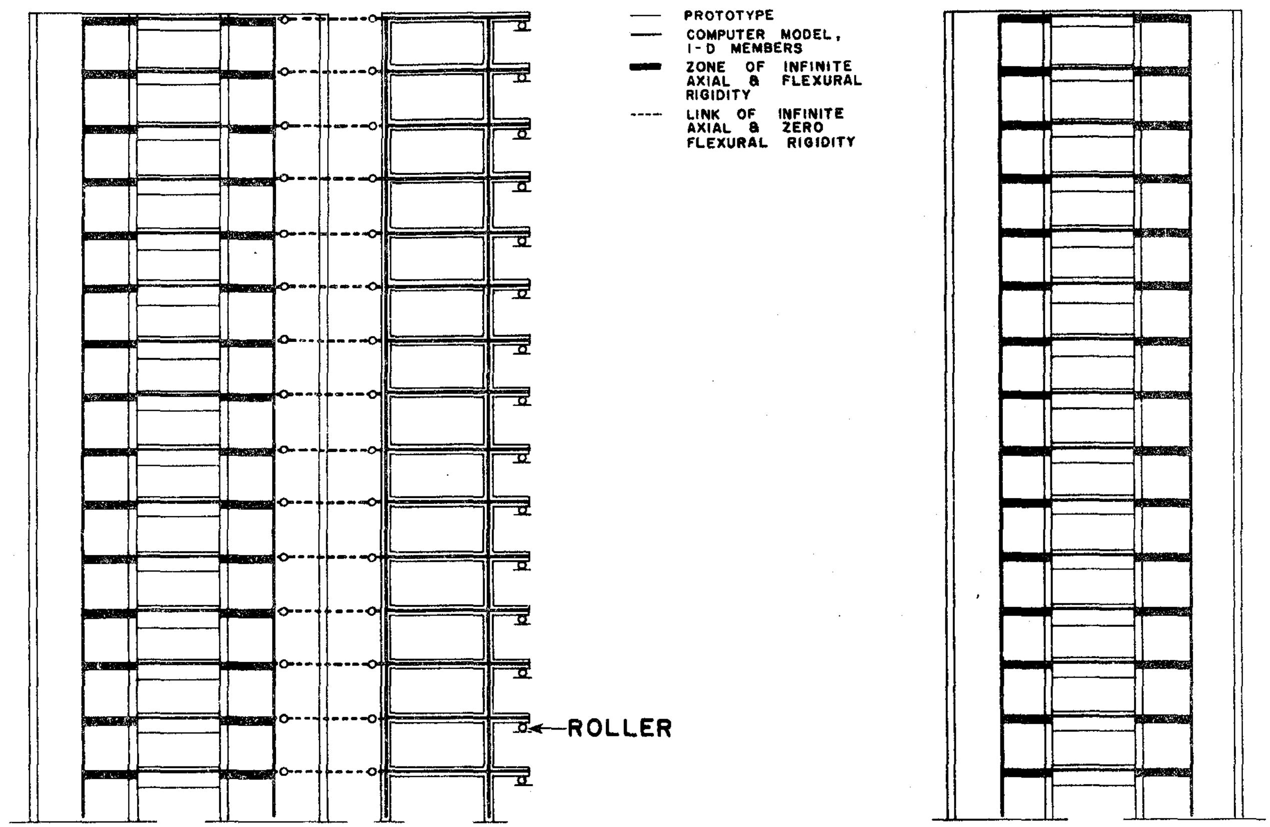


FIG. 6 COUPLING GIRDER DESIGN BY UBC-73 PROVISIONS



(a) Columns (b) Beams

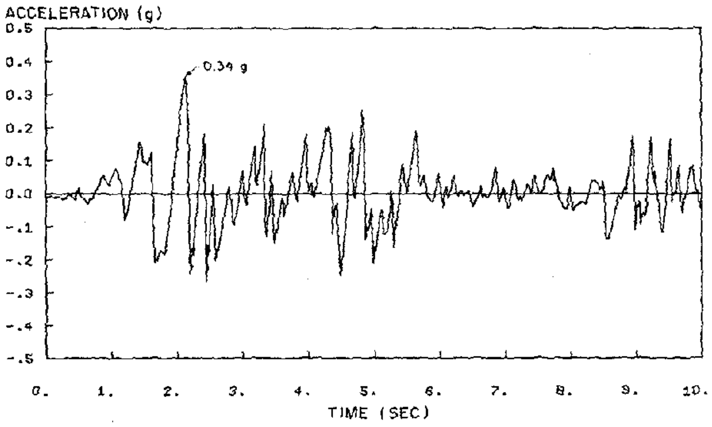
FIG. 7 FRAME ELEMENTS DESIGNED BY UBC-73 PROVISIONS



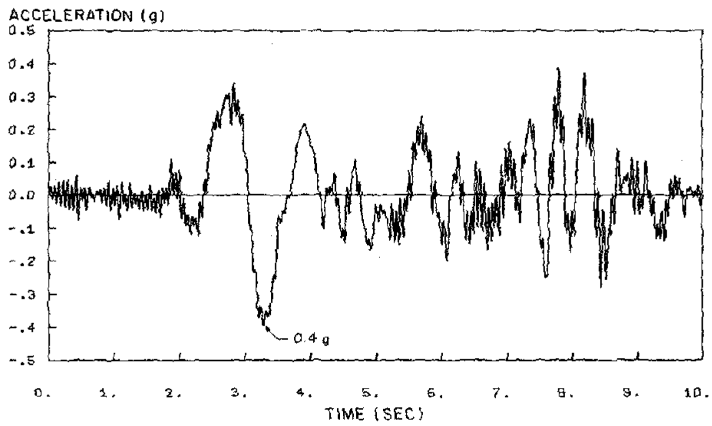
(a) COUPLED WALL - FRAME

(b) ISOLATED COUPLED WALL

FIG. 8 ANALYTICAL MODELS CONSIDERED IN THE ANALYSES



(a) N-S COMPONENT OF EL CENTRO 1940 EQ



(b) DERIVED PACOIMA DAM RECORD

FIG. 9 GROUND ACCELERATIONS CONSIDERED IN THE ANALYSES

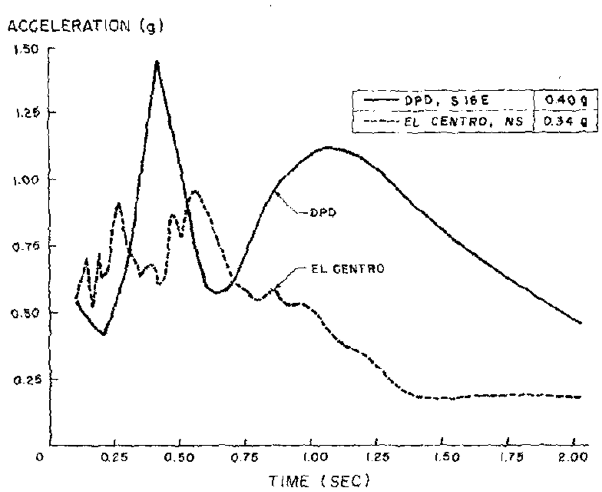
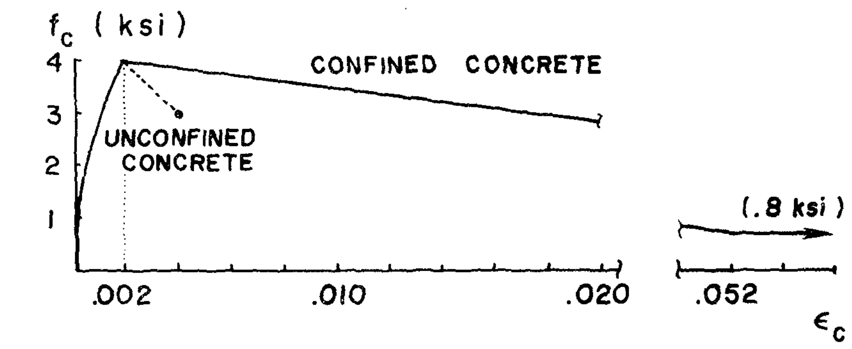
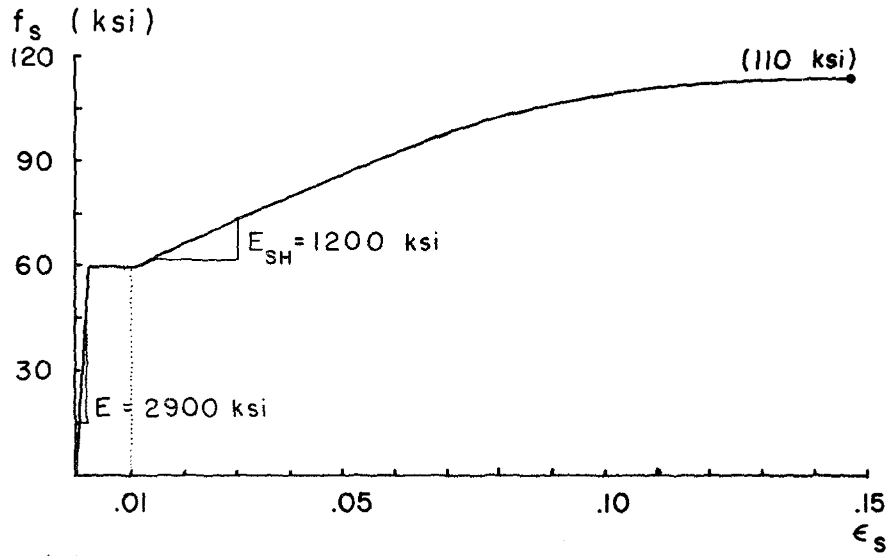


FIG. 10 LINEAR ACCELERATION RESPONSE SPECTRA FOR EL CENTRO AND DERIVED PACOIMA DAM GROUND ACCELERATION RECORDS FOR $\xi = 5\%$



(a) CONFINED AND UNCONFINED CONCRETE



(b) REINFORCING STEEL

FIG. 11 ASSUMED STRESS-STRAIN RELATIONSHIPS FOR THE STRUCTURAL MATERIALS

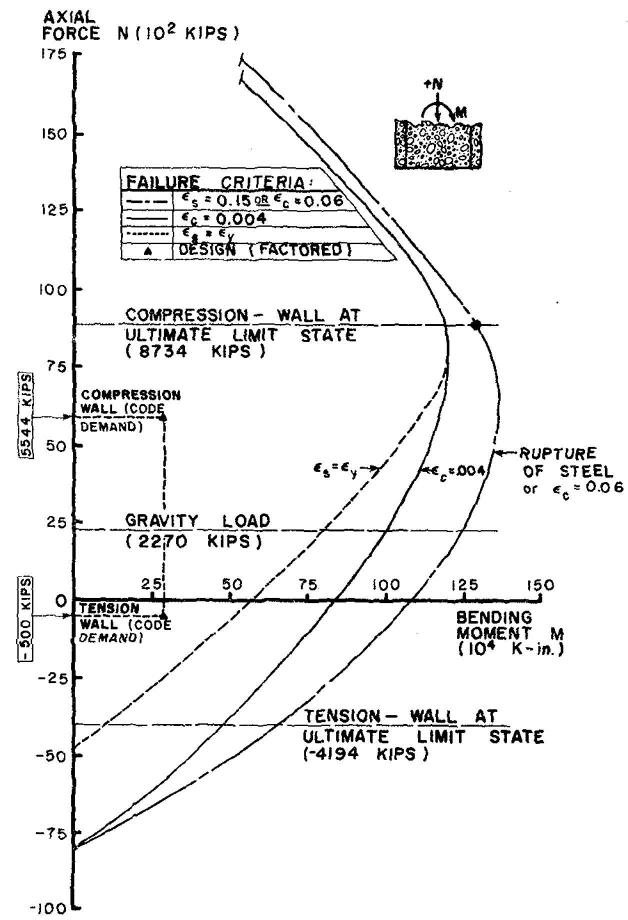


FIG. 12 AXIAL-FLEXURAL STRENGTH OF WALL CROSS SECTION

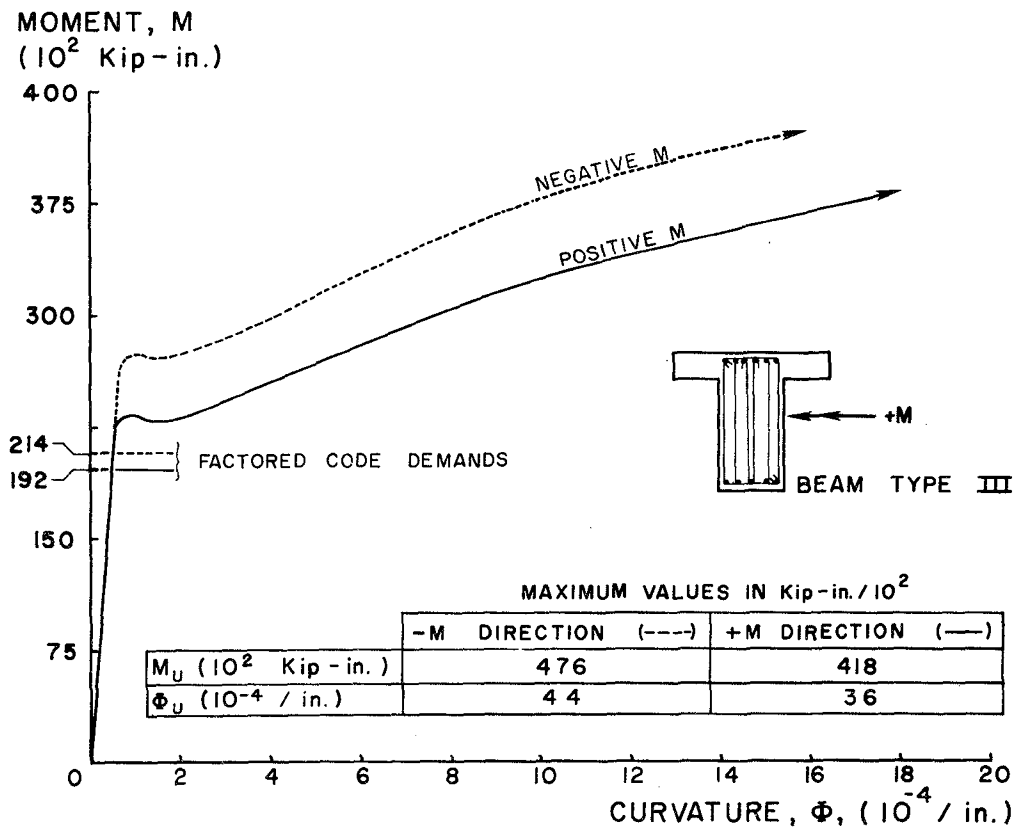


FIG. 13 MOMENT-CURVATURE RELATIONSHIPS OF COUPLING GIRDER (TYPE III)

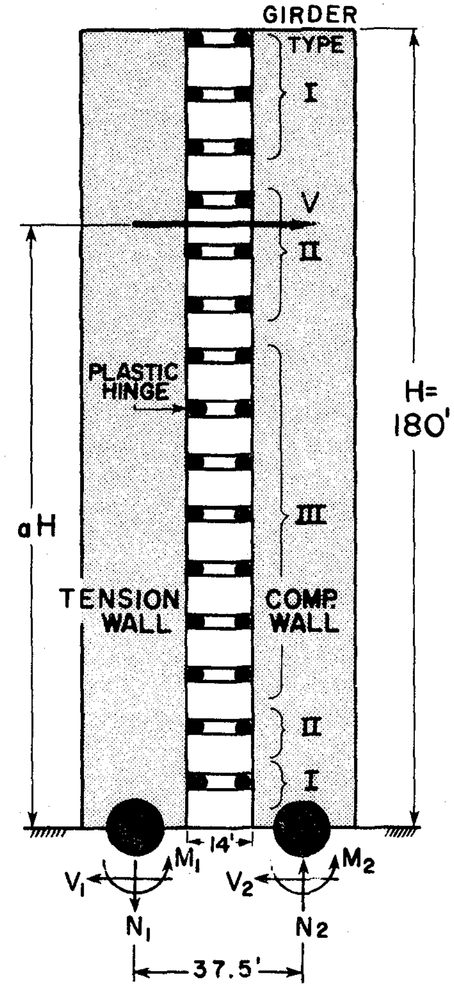


FIG. 14 ASSUMED COLLAPSE MECHANISM FOR THE COUPLED WALL SYSTEM

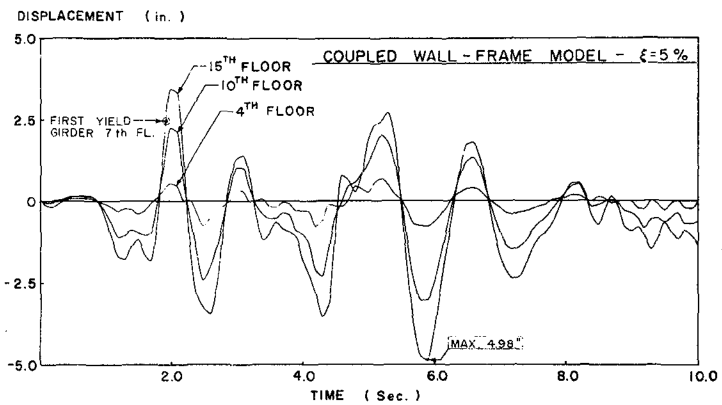


FIG. 15 EL CENTRO RESPONSE FOR ξ=5%: LATERAL DISPLACEMENTS OF 4th, 10th, AND 15th FLOORS

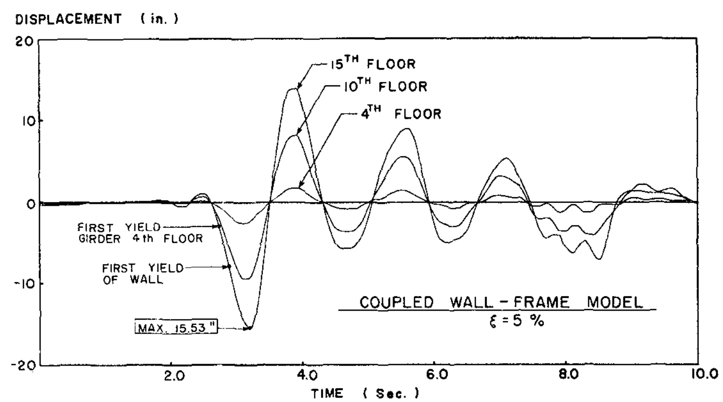


FIG. 16 PACOIMA RESPONSE FOR ξ=5%: LATERAL DISPLACEMENTS OF 4th, 10th, AND 15th FLOORS

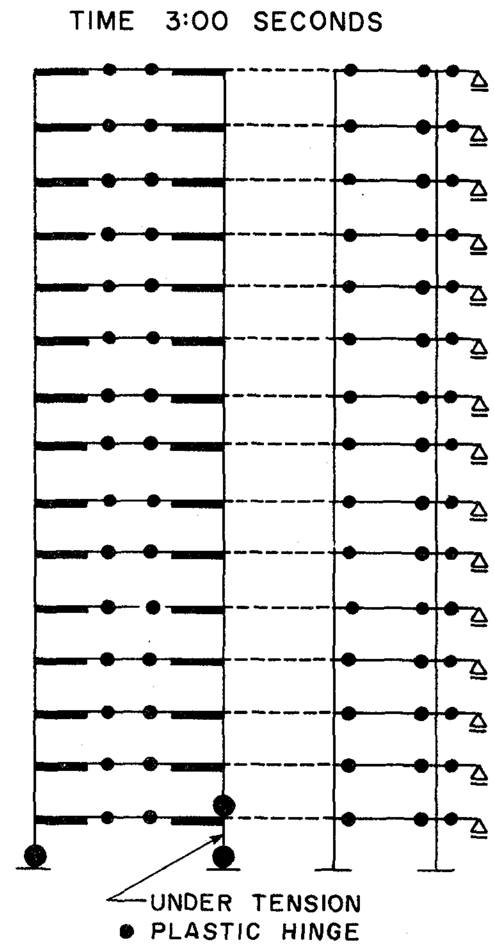


FIG. 17 THE PLASTIC HINGE PATTERN AT 3.00 SEC. OF PACOIMA RESPONSE

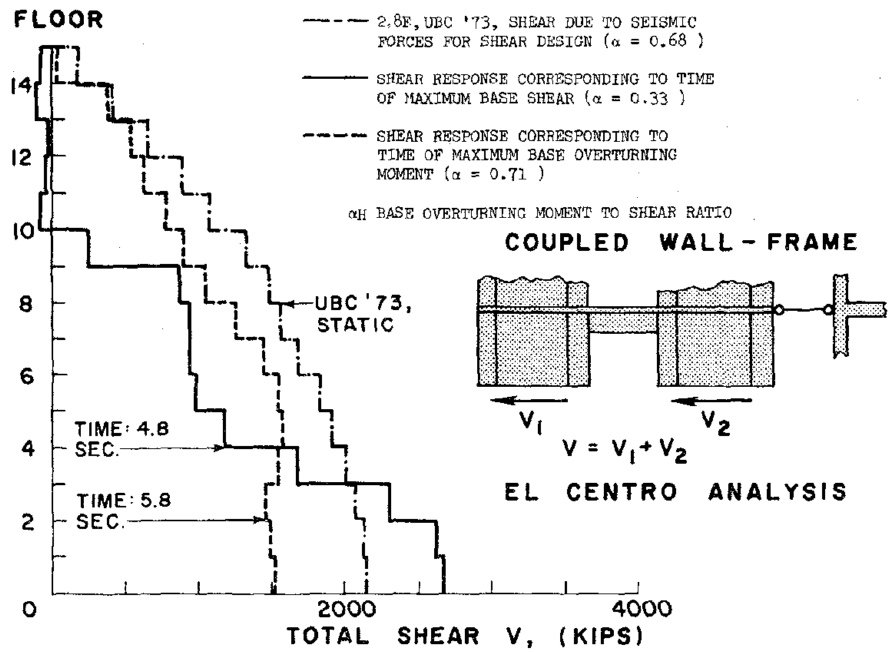


FIG. 18 SEISMIC FORCE AND SHEAR DISTRIBUTIONS ALONG HEIGHT OF COUPLED WALLS ASSUMED BY CODE AND OBTAINED DURING EL CENTRO RESPONSE WITH ξ=5%

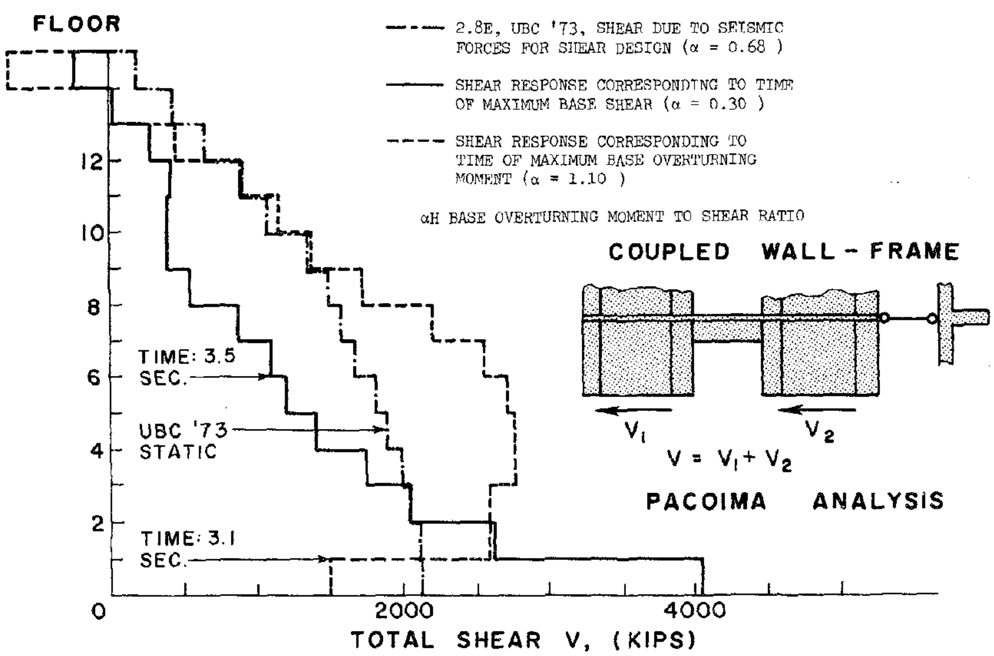


FIG. 19 SEISMIC FORCE AND SHEAR DISTRIBUTIONS ALONG HEIGHT OF COUPLED WALLS BY CODE AND OBTAINED DURING PACOIMA RESPONSE WITH ξ=5%

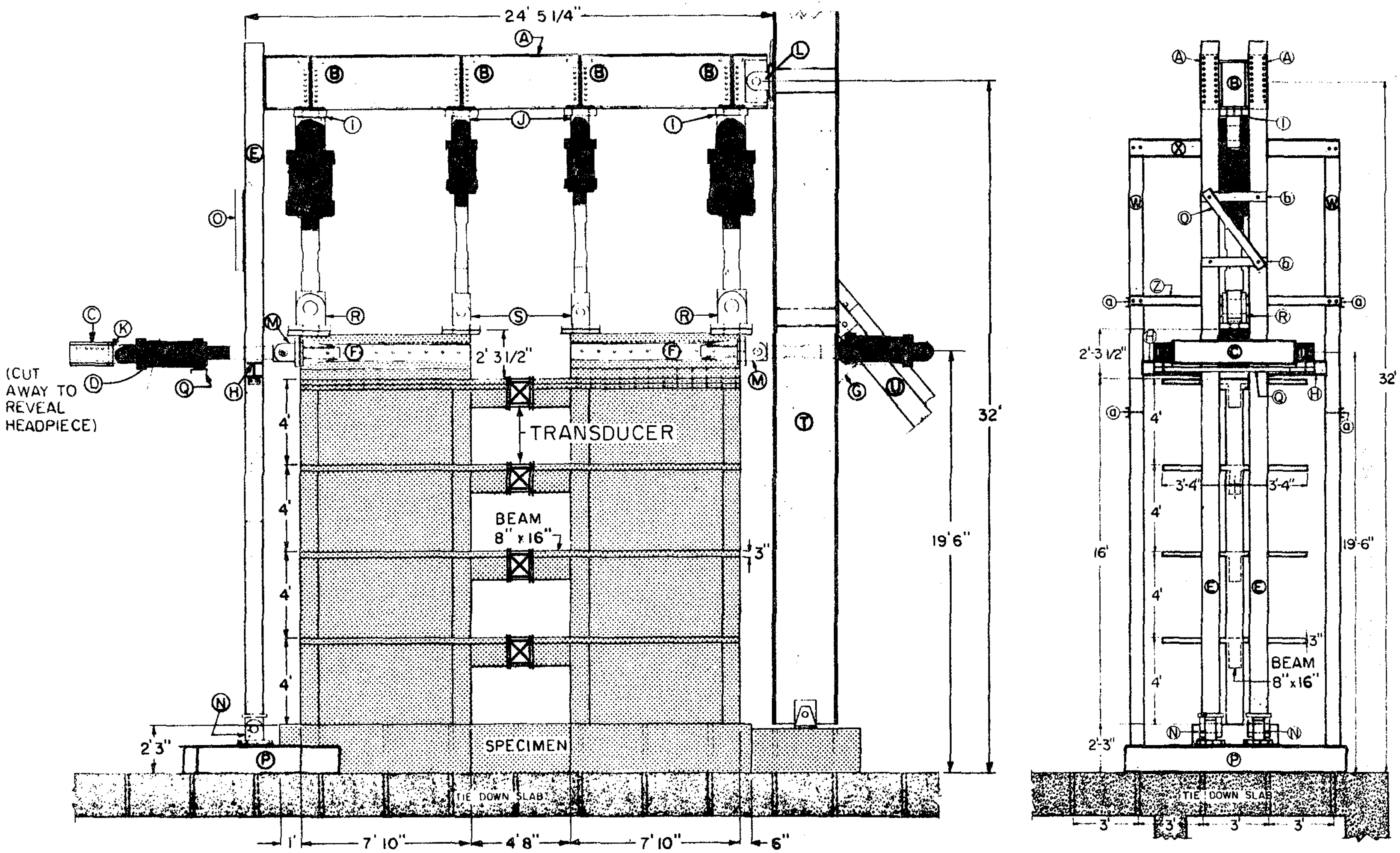


FIG. 20 TEST FACILITY AND 4-1/2 STORY 1/3-SCALE MODEL SUBASSEMBLAGE

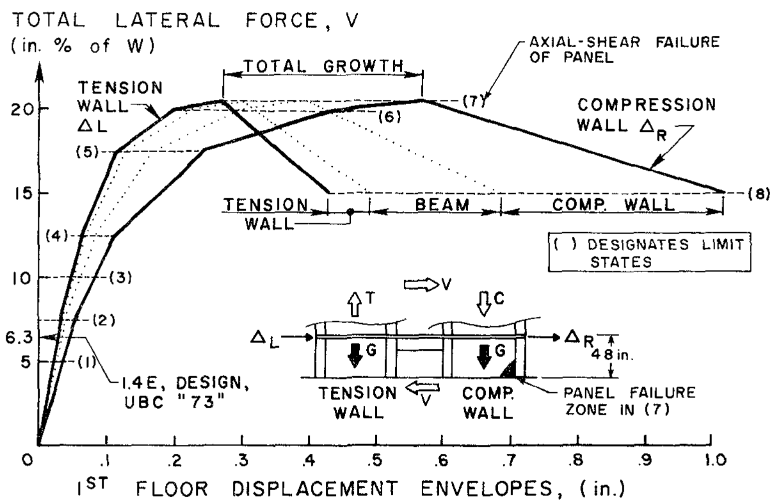


FIG. 21 FIRST FLOOR FORCE-DISPLACEMENT RESPONSE ENVELOPES OF THE COUPLED WALL SYSTEM

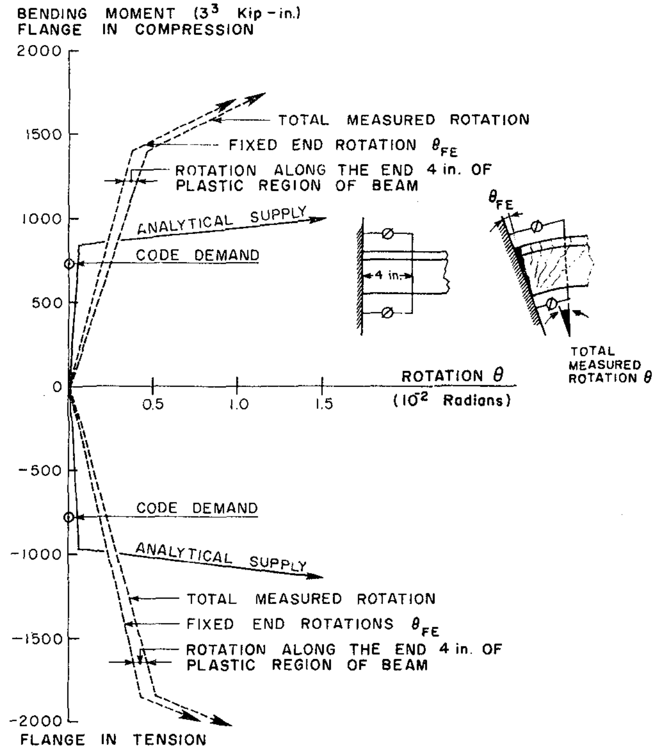


FIG. 22 MOMENT-ROTATION RELATIONS OF THE COUPLING GIRDER

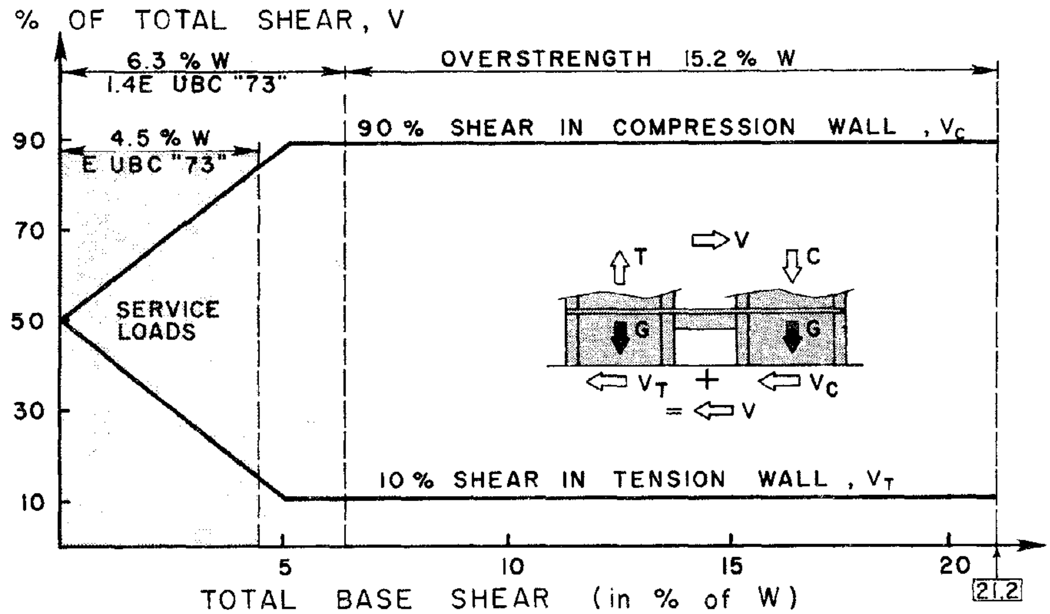


FIG. 23 MEASURED REDISTRIBUTION OF TOTAL BASE SHEAR

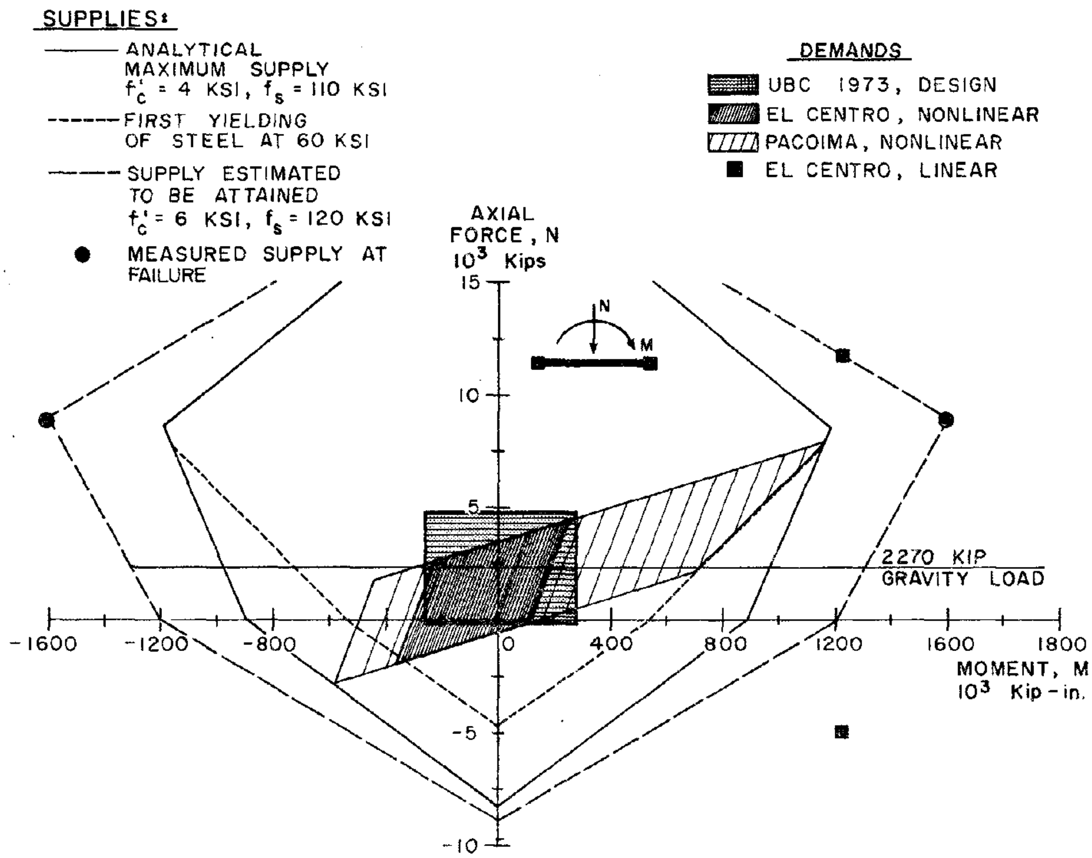


FIG. 24 AXIAL-FLEXURAL DEMANDS AND SUPPLIES AT THE BASE OF ONE WALL

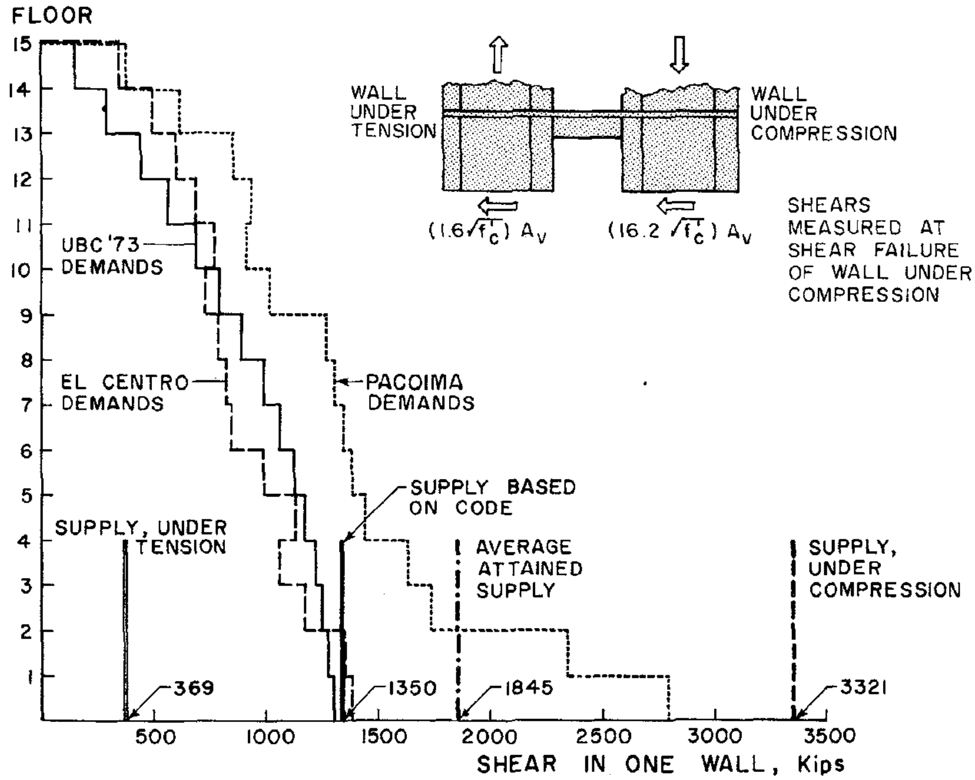


FIG. 25 SHEAR STRENGTH DEMANDS AND SUPPLIES FOR ONE WALL

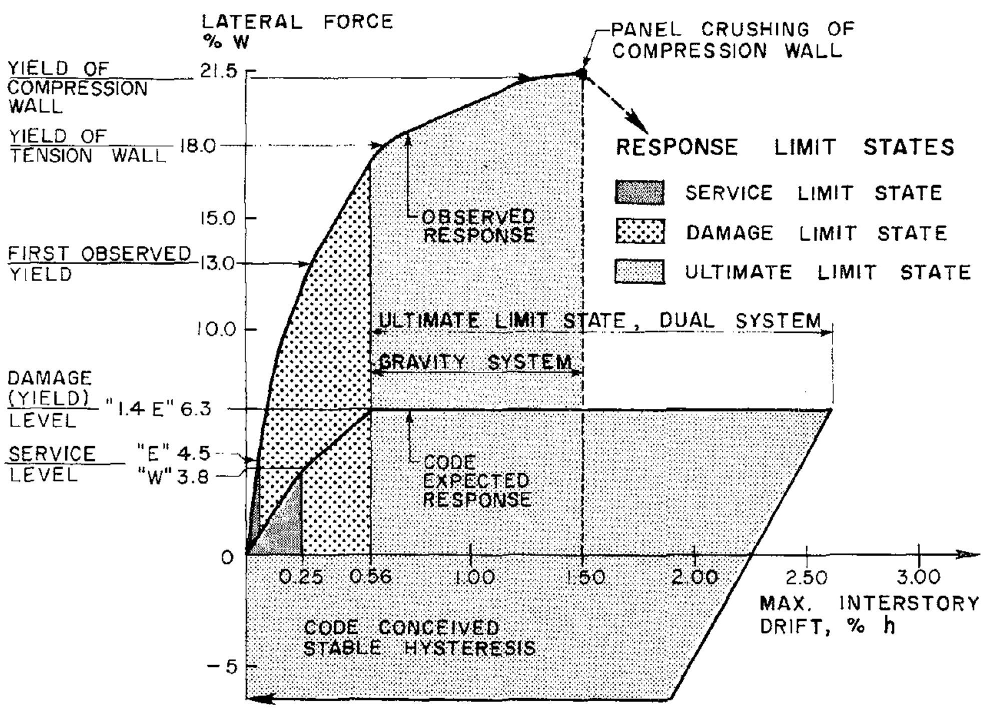


FIG. 26 COMPARISON OF EXPERIMENTAL RESPONSE WITH THE RESPONSE ASSUMED BY THE CODE

EARTHQUAKE ENGINEERING RESEARCH CENTER REPORTS

NOTE: Numbers in parentheses are Accession Numbers assigned by the National Technical Information Service; these are followed by a price code. Copies of the reports may be ordered from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia, 22161. Accession Numbers should be quoted on orders for reports (PB --- ---) and remittance must accompany each order. Reports without this information were not available at time of printing. The complete list of EERC reports (from EERC 67-1) is available upon request from the Earthquake Engineering Research Center, University of California, Berkeley, 47th Street and Hoffman Boulevard, Richmond, California 94804.

UCB/EERC-77/01 "PLUSH - A Computer Program for Probabilistic Finite Element Analysis of Seismic Soil-Structure Interaction," by M.P. Romo Organista, J. Lysmer and H.B. Seed - 1977 (PB81 177 651)A05

UCB/EERC-77/02 "Soil-Structure Interaction Effects at the Humboldt Bay Power Plant in the Ferndale Earthquake of June 7, 1975," by J.E. Valera, H.B. Seed, C.F. Tsai and J. Lysmer - 1977 (PB 265 795)A04

UCB/EERC-77/03 "Influence of Sample Disturbance on Sand Response to Cyclic Loading," by K. Mori, H.B. Seed and C.K. Chan - 1977 (PB 267 352)A04

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