STATE OF THE ART AND PRACTICE IN SEISMIC RESISTANT DESIGN OF R/C FRAME-WALL STRUCTURAL SYSTEMS

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SUMMARY

The state of the practice in the seismic resistant design of R/C frame-wall structural systems is reviewed by analyzing the performance of buildings designed according to current seismic codes. The state of the art in the seismic resistant design of such structural systems is judged by comparing analytically predicted response to experimentally measured behavior. It is concluded that present U.S. seismic codes can lead to poor design and that the present state of the art in modeling R/C frame-wall structures is inadequate. Changes most urgently needed to improve both states are indicated.

INTRODUCTION

Although the last two decades have witnessed marked improvement in the analysis and seismic resistant design of moment-resisting space frames (DMRSF), the ability to analyze and particularly to design frame-wall structural systems has not been comparably improved. For buildings that exceed ten stories, the use of a dual DMRSF-wall system seems to have great potential in seismic resistant design (Ref. 1).

As with any design, the aim of seismic resistant design should be to optimize the resulting building. Several years ago, the author and his associates developed an automated optimal seismic resistant design for DMRSF (Ref. 2). Despite considerable research on the seismic behavior of R/C wall structures (Refs. 1 & 3), the state of the art in quantifying the design (performance) criteria for the seismic resistant design of frame-wall structures remains limited, and a high degree of uncertainty persists in the establishment of design earthquakes and of demands, and in the provision of the supplies of the components of this type of structural system.

The author believes that these limitations and uncertainties remain of such significance as to deter research on the development of a realistic mathematical formulation for the optimum design of such structures. Efforts should be directed towards improving the understanding of the seismic behavior of this type of structural system and the application of this understanding to what the author has defined as *conceptual design.* The basic ideas on which conceptual design is based are: (1) to control and/or decrease seismic demands and (2) to increase the supplies (particularly the energy dissipation) as much as economically feasible. In this sense, the research recently conducted has been significant in improving the understanding of seismic behavior and therefore in identifying the inadequacies and shortcomings in the seismic resistant design provisions of current building codes and in improving application of conceptual design. These advances have motivated this paper, which has the following objectives: (1) to review the states of the practice and of the art of the seismic resistant design of R/C frame-wall structural systems in the U.S. in light of recent research results; and (2) to formulate recommendations for improving the state of the practice as well as that of the art.

STATE OF THE PRACTICE

The design and construction of most seismic resistant buildings in practice generally follow code design provisions. In Refs. 4 and 5, the author has examined in detail present U.S. seismic code provisions, particularly those specified in the *Uniform Building Code* (UBC) (Ref. 6). Here, the soundness of this code will be judged by analyzing the overall performance of UBC-designed structures, rather than by analyzing in detail individual code provisions.

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Soundness of UBC Seismic Resistant Design Procedure for R/C Frame-Wall Structures

The performance of code-designed buildings in recent earthquakes has been studied and described in detail in several publications (Refs. 4 & 7), and the following observations have been made: (1) The maximum lateral resistance (strength) of code-designed structures, especially R/C wall structures, is typically higher (usually by more than a factor of 2) than that required by code; (2) Despite this observed greater strength, many code-designed buildings collapse or suffer serious nonstructural or structural damage that constitutes failure. It can be concluded that building design following present seismic code regulations does not guarantee safety against collapse or failure (unacceptable damage); (3) While buildings based on moment-resisting frames usually collapse through pancaking of one or more stories, this type of collapse is not usually observed in buildings with shear walls that continue to the foundation.

Summary of Results of Research Conducted on UBC-Designed Reinforced Concrete Frame-Wall Structures. The overstrength of wall structures has long been observed (Ref. 5). The two specific examples of UBC-designed structural wall systems discussed below confirm this overstrength and permit the soundness of present UBC design procedures to be assessed by comparing overall responses as expected from the code (state of the practice) with analytically predicted behavior using available computer programs for linear and nonlinear dynamic analyses (state of the art) and with experimentally determined responses.

U.S.-Japan Seven-Story R/C Frame-Wall Structure. A full-scale model of the building structure shown in Fig. 1 was tested in Tsukuba, Japan. This building was designed according to the 1979 UBC and Japanese codes.

UBC Expected Response. The relationship between total lateral force, V, and roof displacement, δ_r, according to UBC minimum requirements is shown as curve *OABC* in Fig. 2(a).

Analytically Predicted Response. Before testing the full-scale model, analyses were conducted using available computer programs. Chavez (Ref. 8) predicted a maximum base shear of 661 kips, when the structure was subjected to the N-S component of the Miyagi-Oki (MO) earthquake record normalized to a peak acceleration of 0.36g. This result is indicated in Fig. 2(a). Charney and Bertero (Ref. 9) estimated the responses shown in Fig. 2(a). When a triangular distribution of lateral force was assumed, the static maximum base shear was 664 kips (Fig. 2(a)). Where a rectangular distribution of lateral force was assumed, the static maximum base shear was 819 kips. From the nonlinear dynamic analysis conducted using the computer program DRAIN-2D, it was computed that under the MO record the maximum base shear would be 767 kips, i.e. 16% greater than that predicted by Chavez and between the two statically predicted maximum base shear values (664 and 819 kips), but closer to that predicted assuming a rectangular distribution of lateral force.

Experimentally Measured Response. (1) The full-scale model of the building was subjected to two series of pseudo-dynamic tests in Japan (Ref. 10). In the first series of tests, the model was considered a single-degree-of-freedom system and tested under an inverted triangular distribution of load. The envelope of hysteretic behavior is shown in Fig. 2. The maximum base shear was 954 kips, i.e. three-and-one-half times greater that the UBC-expected lateral resistance as controlled by flexural yielding. This value was 44% and 25% larger than the value predicted analytically by Chavez and Charney and Bertero, respectively, which clearly indicates that the state of the art in predicting mechanical behavior should be improved.

The full-scale model did not fail during the pseudo-dynamic test in which the inverted triangular lateral load distribution was used, although the displacement ductility considering initial yielding of the wall was more than 6 and the effective displacement ductility of the structure as a whole was close to 4. This high observed ductility arises primarily because under the inverted triangular load distribution, the wall and the beams and columns of the frame were subjected to small shear stress. The wall was designed for a unit nominal shear stress, v_u, smaller than $5\sqrt{f_c'}(psi)$ and at the maximum measured $V = 954$ *kips*, v_u was smaller than $12\sqrt{f_c'}(psi)$. The experimental results confirmed the author's previous conclusions: If by proper (conceptual) design, it is possible to keep shear stresses low, R/C shear wall structures can offer sufficient dissipation of energy (ductility and stable hysteretic behavior) through flexural yielding to resist even the effects of extreme ground shaking.

From comparison of the analytically predicted curves and the experimental envelopes, it can be seen that the structure was considerably stiffer than expected from the UBC and analytically predicted stiffnesses. The full-scale model, after being subjected to the first series of tests with the inverted triangular distribution of lateral load, was repaired. 'Nonstructural' spandrel walls and partitions were also added in all stories except the first. The model was then subjected to a series of tests similar to the first series. Finally, the model was loaded statically using a rectangular distribution of lateral load. As illustrated in Fig. 2(b), the structure under this load was able to resist a maximum base shear of 1315 kips, or 4.8 times the UBC-required design ultimate lateral strength. However, when a base shear of 1315 kips was reached at a lateral displacement of 11.3 in., the wall failed in shear at the first story with a sudden drop in resistance. Note that this displacement was about 16% smaller than that obtained in the first series of tests without failure. Comparison of results from the two test series indicates the importance of the type of loading and the added 'nonstructural' elements to the behavior of frame-wall structures. While resistance may be considerably increased, ductility may be jeopardized.

A 1/5th scale model of the building, shown in Fig. 1, was subjected to several series of tests at the Earthquake Simulator facility at Berkeley, and in Fig. 2 the force-deformation envelope obtained from these tests is compared with those obtained pseudo-dynamically on the full-scale model. As can be seen, the envelope obtained from the 1/5th scale model shows a significant increase in strength when compared with results obtained for the full-scale model using a triangular distribution of lateral force.

Frame-Coupled Wall Structural System. To assess whether R/C frame-coupled wall structural systems designed according to the UBC possess desirable characteristics at all primary limit states and, particularly, at ultimate limit states (damageability and safety against collapse) under the maximum expected earthquake ground motion, Aktan and Bertero (Ref. 3) conducted analytical and experimental studies on the fifteen-story building system illustrated in Fig. 3. This building was designed according to 1973 UBC, 1979 UBC, and ATC-3 provisions.

The 1973 UBC-designed structure was thoroughly analyzed using linear and nonlinear static and dynamic computer programs (Ref. 3). A four-and-one-half story, one-third scale model subassemblage of one coupled wall of the structural system was constructed and tested. In Fig. 4, the code-expected total lateral load-maximum interstory drift values for the coupled wall are compared to those obtained experimentally. The contrast in these values is striking. The stiffness and strength of the coupled wall are observed to exceed significantly the minimum values required by the code. The capacity to dissipate energy is, however, not as large as would be expected according to the strength and the code-assumed displacement ductility. Therefore, the following questions arise: (1) Why is the experimentally observed strength greater (3.4 times) than the strength expected from the code? (2) Why is the measured displacement ductility smaller than expected? (3) What are the possible consequences of this overstrength and relatively small ductility?

The observed overstrength was primarily due to the higher axial-flexural strengths of the coupling girders and of the wall subjected to higher compression. A detailed discussion of the reasons for these observed higher axial-flexural strengths is contained in Ref. 3. A summary of these reasons follows.

The overstrength of the coupling girder axial-flexural strength was caused by: the larger yielding strength and ultimate capacity of reinforcing steel; the compressive axial force in these girders which arose from the shear redistribution in walls; the hypothetical underestimated slab contribution (effective flange width and contribution of the slab steel) as opposed to the actual contribution; and the relatively low nominal shear stress for which these girders were designed ($v_u = 6.4\sqrt{f_c'(psi)}$ versus $10.0\sqrt{f_c'(psi)}$ allowed by the UBC).

The coupling girders developed higher shear due to their higher axial-flexural capacity.

The walls were consequently subjected to axial forces significantly higher than those anticipated by the code. Higher compression resulted in higher axial-flexural resistance. Lower compression or even tension resulted in lower wall flexural capacity. The flexural resistance of the walls under compression and tension contributed 34% and 6%, respectively, to the total moment-overturning capacity.

The total contribution of the axial-flexural strength of the walls to the total overturning moment and, therefore, to the total shear resistance of the structural system was higher than expected due to: larger yield and maximum strength of the actual reinforcing steel bars as opposed to code-specified values; increase in compressive strength of the concrete used; and the enhancement of this strength in the edge members by the high degree of confinement. The axial-flexural strength attained in the compression wall was approximately 6.0 times greater than the code design demand and 1.4 times greater than the analytically predicted supply. Furthermore, the walls were able to develop these higher axial-flexural strengths without failing prematurely in shear because their wall panels were designed for a maximum $v_u = 6.1\sqrt{f'_c(psi)}$ (this value includes the $2.8E$, where E represents earthquake load capacity, and the capacity reduction factor $\Phi = 0.85$) which is less than the UBC-allowed factor of $10.0\sqrt{f'_c(psi)}$.

The significant increase in total shear force that had to be resisted by the coupled walls as a consequence of the actual provided axial-flexural strength was one of the main reasons for the observed relatively small ductility. An even more important reason, perhaps, was the relative distribution of this total shear between the two walls of the coupled system. The typical measured redistribution of shear force at the base of the coupled wall model during a half-cycle of loading is illustrated in Fig. 5 (Refs. 3 & 5). The wall under compression attracted 85% of the total base shear even at the service load level as defined by the 1973 UBC. At the ultimate limit states, the compression wall resisted 90% of the total shear. The nominal shear stresses, v_{max}, in the two walls at the failure of the panel of the wall under compression, amounted to $1.6\sqrt{f'_c(psi)}$ and $16.2\sqrt{f'_c(psi)}$ for the walls in tension and compression, respectively.

Because of the high nominal shear stress developed in the compression wall, $16.2\sqrt{f'_c(psi)}$, it is not surprising that once the maximum lateral resistance capacity was reached (Fig. 4), this wall was not able to undergo large inelastic deformation. The somewhat sharp drop in strength after a relatively short plastic plateau is attributable to panel crushing as a consequence of the high axial compression and shear that developed at the critical region of the compression wall when it began to yield in flexure. Tests conducted on similar walls but under smaller axial and shear force have shown higher ductility and larger energy dissipation capacity, although yielding occurred at a slightly lower total lateral force (Ref. 1).

Concluding Remarks Regarding State of the Practice

The results presented in Figs. 2, 4, and 5 and the above discussion indicate that although UBC-designed structures have a supplied axial-flexural strength significantly higher than the UBC-required strength, the shear forces developed at the critical regions of structures when the yielding axial-flexural capacity is reached may exceed the available shear strength. A semibrittle failure may consequently occur before the wall will be capable of supplying the required energy dissipation capacity. It can therefore be concluded that the UBC seismic provisions and design procedures do not guarantee a sound seismic resistant design of frame-walled structures. The seismic design forces are specified at a fictitious level and the recommended linear elastic analysis procedure cannot predict the actual response under the probable extreme earthquake ground motion. The designer is not properly guided in detailing structures for actual behavior (depicted in Figs. 2 and 4 by the experimental curves), because he/she is required to make computations only and therefore to think and/or visualize seismic behavior according to the code-conceived linear elastic response represented in Figs. 2 and 4 by the line OA.

The large axial-flexural overstrength results from a series of code requirements: in the case of dual bracing systems, walls alone must resist the total shear; the edge (vertical boundary) members along the shear walls must be designed not only to carry all vertical stresses resulting from all gravity loads and the total horizontal forces acting on the wall, but also to account for a capacity reduction factor, Φ, equal to 0.70 or 0.75; neglect or underestimation of the contribution of floor systems (R/C slabs) to the lateral stiffness and strength, particularly the contribution of the slab reinforcement to the girders' negative end moments; neglect of increases in strength due to the strain hardening of reinforcing bars, which becomes increasingly significant as the required ductility of the structural system increases. For the cases presented in Figs. 2 and 4, another main reason for the observed axial-flexural overstrength was that the strengths of the materials (the $f_c^{'}$ and f_y of the concrete and steel, respectively) were considerably higher than the specified values.

Changes in the UBC seismic provisions are urgently needed. Design against shear failure should be based on the maximum shear that can be developed in structural members according to the actual strength supplied to a building and the actual distribution of total shear among structural members. Numerous suggestions for change needed in the UBC to improve the state of practice have been proposed in Ref. 5.

STATE OF THE ART

Detailed discussion of the state of the art in the seismic resistant design of buildings with particular emphasis on frame-wall structures is provided in Refs. 3-5. In Ref. 4, the author concludes that "despite advances there are many uncertainties involved in: (1) predicting the dynamic characteristics of future earthquakes; (2) modeling the dynamic characteristics of the building-soil foundation system; (3) estimating the actual supplied strength and deformation capacity of buildings; and (4) conducting reliable numerical analyses of the response of mathematical models and therefore in estimating the demands on structures. There is an urgent need to increase our present knowledge through integrated educational, research, and development efforts."

To illustrate deficiencies in the present state of the art, correlations between analytically predicted and experimentally measured behavior of the frame-wall structure shown in Figs. 1 and 3 are briefly discussed below.

U.S.-Japan Seven-Story Reinforced Concrete Wall Structure.

The analytical responses for the full-scale reinforced concrete wall structure did not correlate well with the experimental results (Fig. 2). A primary reason for this discrepancy is the way in which frame-wall structures are modeled. The current modeling technique for a frame-wall system such as that illustrated in Fig. 1 is to consider only in-plane action, using an analytical model similar to that shown in Fig. 6(a) which can be considered a pseudo-three-dimensional model. It is assumed that at the ultimate limit state of collapse, the structure dissipates energy by deforming according to the mechanism illustrated in Fig. 6(b). Note that in this model the shear wall is modeled as a column located at the axis of symmetry of the wall and therefore rotating about the wall centroidal axis at the base. Furthermore, the transverse coupling between the interior frame-wall B and the exterior frames A and C (see Fig. 1) is ignored.

The experiments described here have shown that after yielding of the main shear wall, the behavior of the structure was as illustrated in Fig. 7, i.e. the wall rotated (rocked) practically around the center of its edge member and the three-dimensional frame surrounding the wall outriggered it, restraining its rocking and therefore increasing the axial compressive force acting on it and consequently increasing its flexural resistance. The migration of the longitudinal neutral axis of the wall, the rocking of the wall, and the restraint of this rocking by the outriggering action of the frame surrounding the wall significantly increased the lateral resistance capacity of the three-dimensional structure. For a detailed evaluation of the contribution of the rocking and the outriggering interaction, see Ref. 11. After the experiments on the full-scale model, Kabeyasawa *et al.* (Ref. 12) developed a mathematical model which incorporated these new features. The predicted response correlated very well with the observed response. The importance of integrated experimental and analytical research is thus underscored.

Frame-Coupled Wall Structural System

Code design procedures and most computer programs for linear analyses of structural systems with identical coupled walls assume that the lateral stiffness of these walls is the same and remains constant during "linear elastic response." For two similar walls that are coupled, each wall is assumed to resist, and is therefore designed for, half of the total shear resisted by the coupled walls. The lateral stiffness (flexural and shear) of R/C structural elements (particularly walls), is sensitive to the amount of axial force. Therefore, the stiffness of the two coupled walls and consequently the amount of shear resisted by each cannot be the same since as a result of the coupling girders, the axial force acting in each coupled wall will start to differ as soon as a lateral force is induced. The difference must increase as the lateral force increases. This has been clearly proved experimentally (see Fig. 5). The effects of variations of axial force on lateral stiffness (flexural and shear) have been studied experimentally in connection with studies conducted on the 1/5th scale model shown in Fig. 1(c) (Ref. 11) and in recent experiments on walls similar to those shown in Fig. 3 which clearly show the tremendous influence of axial force on the lateral flexural and shear stiffness of the wall.

Concluding Remarks Regarding the State of the Art

Sophisticated computational methodologies have been developed for the analysis of three-dimensional mathematical models. *However, the present state of the art of modeling R/C frame-wall building structures does not allow for adequate and/or realistic assessments of force and distortion demands.* The problem is due primarily to the difficulty and uncertainty inherent in assessing axial, shear-flexural, and torsional stiffnesses, strengths, and force-distortion hysteretic capacities of R/C structures. These difficulties and uncertainties are especially acute when the behavior of structural members and/or critical regions is affected by three-dimensional interaction between internal forces and induced distortions. To use computational capabilities efficiently, the state of the art of predicting realistic supplies must first be improved. This improvement can be achieved through *integrated analytical and experimental research on the behavior of buildings or that of realistic three-dimensional physical models* subjected to actual earthquake ground motions or realistic representations thereof.

ACKNOWLEDGMENTS

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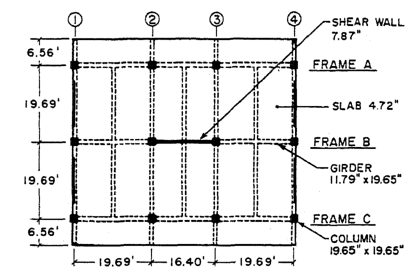
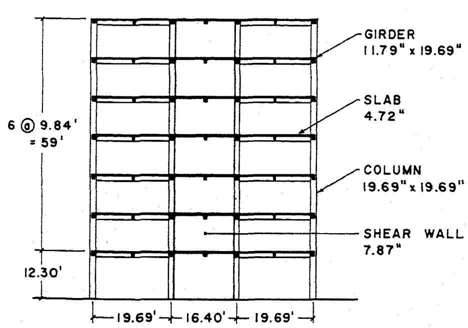
FIGURES

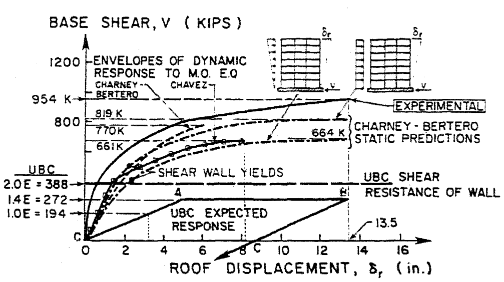


(a) PLAN OF PROTOTYPE

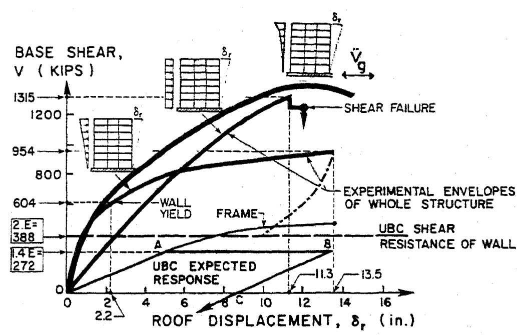


(b) FRAME B OF PROTOTYPE

FIG. 1 SEVEN-STORY REINFORCED CONCRETE FRAME-WALL TEST BUILDING



(a) UBC, EXPERIMENTAL, STATIC & DYNAMICALLY PREDICTED RESPONSES



(b) UBC & EXPERIMENTAL ENVELOPES

FIG. 2 COMPARISON OF UBC, STATICALLY & DYNAMICALLY PREDICTED & EXPERIMENTAL TOTAL LATERAL LOAD-ROOF DISPLACEMENT RELATIONS FOR BUILDING IN FIG. 1

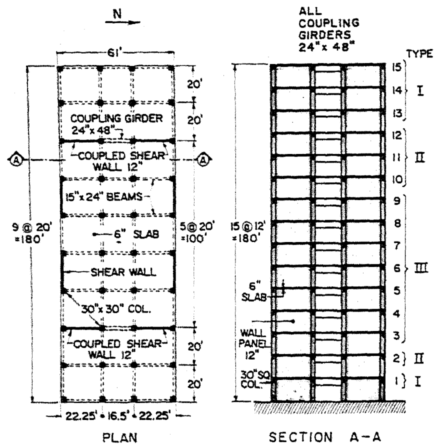


FIG. 3 STRUCTURAL LAYOUT OF 15-STORY R/C FRAME-COUPLED WALL BUILDING

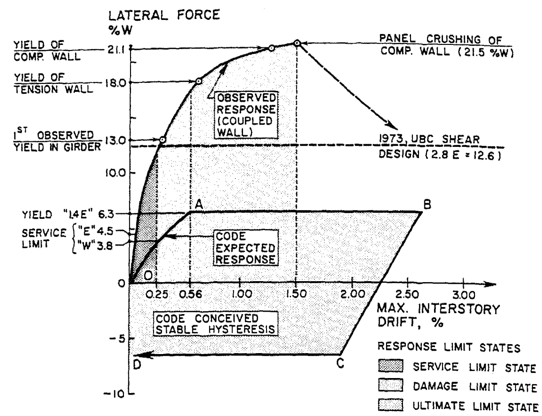


FIG. 4 COMPARISON OF EXPERIMENTAL RESPONSE WITH UBC-EXPECTED RESPONSE OF COUPLED WALL OF BUILDING IN FIG. 3

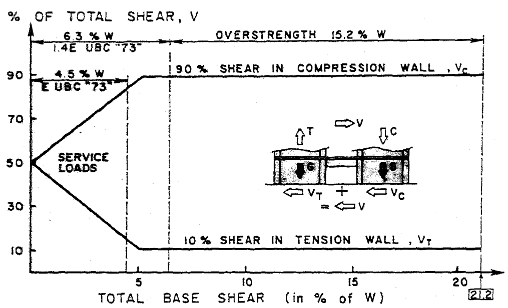
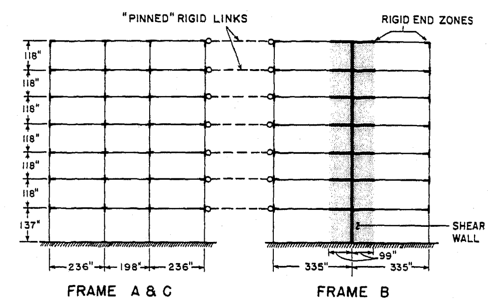
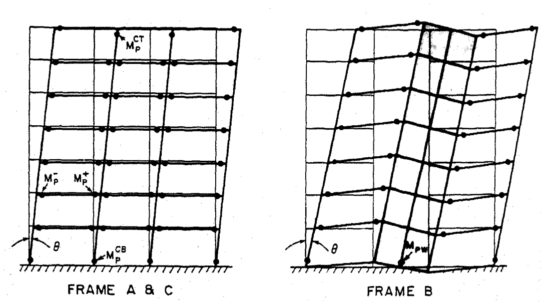


FIG. 5 MEASURED REDISTRIBUTION OF TOTAL BASE SHEAR OF COUPLED WALLS IN BUILDING OF FIG. 3



(a) PSEUDO-THREE-DIMENSIONAL MODEL CONSIDERING ONLY PLANAR ACTION



(b) PLANAR MECHANISM OF MOTION

FIG. 6 ANALYTICAL MODEL FOR FRAME-WALL SYSTEM OF FIG. 1

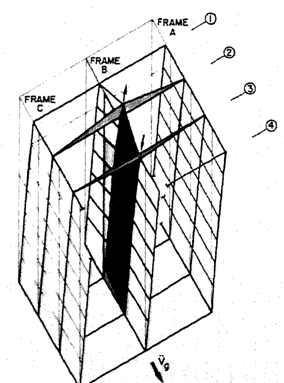


Fig. 7 THREE-DIMENSIONAL INTERACTION OF WALL & FRAMES OF BUILDING IN FIG. 1