DESIGN OF FRAME-WALL STRUCTURES

by

Arnaldo T. Derecho

Partial Proceedings of the Workshop
on ERCBC, July 11-15, 1977
University of California at Berkeley

Portland Cement Association
Research & Development Construction
Technology Laboratories
5420 Old Orchard Road
Skokie, Illinois 60076

50272 -101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NSF/RA-770636	2.	3. Recipient's Accession No. PB 80 116874
4. Title and Subtitle Design of Frame-Wall Structures (Partial Proceedings of the Workshop on ERCBC, July 11-15, 1977, University of California at Berkeley)			5. Report Date July 1977
			6.
7. Author(s) A.T. Derecho			8. Performing Organization Rept. No.
9. Performing Organization Name and Address Portland Cement Association Research and Development Construction Technology Laboratories 5420 Old Orchard Road Skokie, Illinois 60076			10. Project/Task/Work Unit No.
			11. Contract(C) or Grant(G) No. (C) GI43880 (G)
12. Sponsoring Organization Name and Address Engineering and Applied Science (EAS) National Science Foundation 1800 G Street, N.W. Washington, DC 20550			13. Type of Report & Period Covered
			14.

15. Supplementary Notes

16. Abstract (Limit: 200 words)

A brief discussion of the general considerations involved in the earthquake-resistant design of reinforced concrete structures consisting of frames stiffened by structural walls is presented. Particular attention is given to the desirability of having most of the inelastic action occur in elements which are not critical to the overall stability of the structure. Basic requirements for the development of a practical and reliable design procedure usable by the average practising engineer are noted. The need for a comprehensive combined analytical and experimental program to lay the basis for such a development is stressed. A proposed design methodology applicable to isolated structural walls is presented briefly. With appropriate modifications, the same approach can be used for developing a design procedure for frame-wall structures. The performance of frame-wall structures during recent earthquakes has shown that such a system, when properly conceived and designed, provides an efficient solution to the requirements of life safety (i.e., no collapse) and damage control in earthquake-resistant buildings.

17. Document Analysis a. Descriptors

Earthquakes	Earthquake resistant structures	Structural analysis
Elastic waves	Structural design	Concrete construction

b. Identifiers/Open-Ended Terms

Frame-wall structures

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
NTIS		34
	20. Security Class (This Page)	22. Price A03-A01

(See ANSI–Z39.18) See *Instructions on Reverse* OPTIONAL FORM 272 (4–77) (Formerly NTIS–35) Department of Commerce

DESIGN OF FRAME-WALL STRUCTURES

by

Arnaldo T. Derecho
Manager, Structural Analytical Services
Portland Cement Association, Skokie, Illinois

ABSTRACT

A brief discussion of the general considerations involved in the earthquake-resistant design of reinforced concrete structures consisting of frames stiffened by structural walls is presented. Particular attention is given to the desirability of having most of the inelastic action occur in elements which are not critical to the overall stability of the structure.

Basic requirements for the development of a practical and reliable design procedure usable by the average practising engineer are noted. The need for a comprehensive combined analytical and experimental program to lay the basis for such a development is stressed. A proposed design methodology applicable to isolated structural walls is presented briefly. With appropriate modifications, the same approach can be used for developing a design procedure for frame-wall structures.

DESIGN OF FRAME-WALL STRUCTURES

by

Arnaldo T. Derecho
Manager, Structural Analytical Services
Portland Cement Association, Skokie, Illinois

INTRODUCTION

General

Observations of the performance of buildings subjected to earthquakes during the last decade have focused attention on the need to minimize damage in addition to ensuring the general safety of buildings during strong earthquakes [1-3]. The need to control damage to both structural and nonstructural components during earthquakes becomes particularly important in buildings such as hospitals and other facilities which must continue operation following a major disaster. Damage control, in addition to life safety, is also economically desirable in tall buildings designed for residential and commercial occupancy, since the nonstructural components in such buildings usually account for from 60 to 80 percent of the total cost. For the purpose of this discussion, a strong earthquake is that which can reasonably be expected to occur several times during the life of a structure.

While reinforced concrete rigid frame structures have performed reasonably well in earthquakes, particularly with respect to the primary performance criterion of life safety (i.e., no collapse), their inherent flexibility usually results in lateral displacements that cause significant damage to nonstructural components in a building. Reinforced concrete structural walls (or shear walls) have long been used to stiffen tall buildings against wind. When properly designed, walls offer one of the most logical and economical means of minimizing damage in buildings subjected to strong ground motion.

There is little doubt that structural walls offer an efficient way to stiffen a building against lateral loads. When proportioned so that they possess adequate lateral stiffness to limit interstory distortions to acceptable levels and designed to maintain their strength under the earthquake-induced motions, walls effectively reduce the likelihood of damage to the nonstructural elements in a building. When used with rigid frames, walls form a structural system that combines the gravity-load- carrying efficiency of the rigid frame with the lateral-load-resisting efficiency of the structural wall.

In its simplest form, the frame-wall structure consists of an unperforated wall linked to a rigid frame. The linkage may consist of beams rigidly connected to the wall or just the floor slabs. Often, the 'wall' in a frame-wall system takes the form of coupled walls, i.e., walls in the same plane connected by beams. This is typical in the corewalls of so-called 'hull-core' or 'tube-in-tube' systems. As mentioned, the structural walls in frame-wall systems, whether consisting of single unperforated walls or of coupled walls, are generally used in multistory buildings when the stiffness of the frame alone (as designed for gravity loads) is not sufficient to limit the lateral displacements due to wind or earthquake motions to tolerable levels. It is mainly this application of walls in multistory structures which will be discussed here. The behavior of short walls, i.e., walls with a height-to-depth ratio of less than about 2, is governed by slightly different considerations [4] than those applying to tall, relatively slender, walls and will not be discussed here.

Distinguishing Feature

A major distinction between the typical frame-wall system and the rigid frame structure is the interaction that takes place between the frame and the wall under lateral loading (Fig. 1). This interaction, which results from the tendency of the basic elements to deflect in different modes under lateral load, often gives rise to horizontal story shears acting on the frame columns at the top stories which are greater than the corresponding total applied story shears. In the presence of major discontinuities in stiffness, particularly in the wall, this same interactive behavior can result in horizontal story shears acting on the wall and the frame which, separately, can be appreciably greater than the corresponding total applied story shears. This is illustrated in Fig. 2, which shows the horizontal story shears resisted by the wall and the frame columns in a statically loaded frame-wall structure where the wall is discontinued at the first story [5]. The story shear shown as corresponding to the wall at the first story level is actually resisted by the columns supporting the wall. Under strong ground motion, high ductility or deformation requirements tend to be associated with such discontinuities.

Note that horizontal interactive forces due to lateral loads can also occur between coupled walls which have different stiffness distributions along their height, as shown in Fig. 3.

BASIC PLANNING AND DESIGN CONCEPTS

Typical Plan Configurations

The general objective in the design of frame-wall structures for strong ground motions is the provision of sufficient stiffness, strength and deformation capacity to withstand the induced forces and deformations while limiting the overall displacements to acceptable levels. In planning multistory frame-wall structures to meet this objective, certain general features are desirable. Among the more important of these are plan symmetry, the avoidance of significant discontinuities in mass, stiffness or geometry and the location of stiffening elements where they are most effective in resisting displacements parallel to the plan axes as well as torsional motions. This third consideration requires that structural walls be located close to the plan periphery. Because torsion (whether due to the non-coincidence of the centers of mass and resistance or to phase differences in the excitation of various points at the base of a structure by seismic waves propagating at finite speed [6]) can induce significant forces in corner vertical elements, an effort should be made early in the design stage to minimize its effects. The above three basic desirable features are intended to minimize torsional effects and the force concentrations and associated deformation requirements that occur at regions of major discontinuity in a structure. An example of a plan for a rectangular building illustrating the above plan features is shown in Fig. 4.

A frame-wall plan configuration that is commonly used in tall office buildings is the so-called 'hull-core' or 'tube-in-tube' system, consisting of a centrally located service core and a closely spaced grid of frame elements

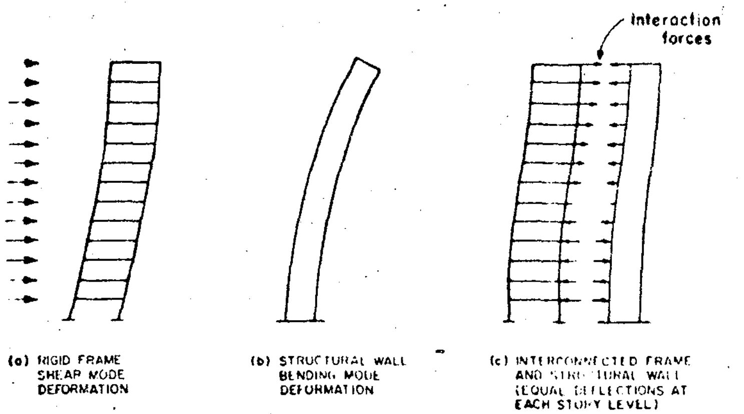

Fig. 1 Structural Wall-Frame Interaction Under Lateral Loading

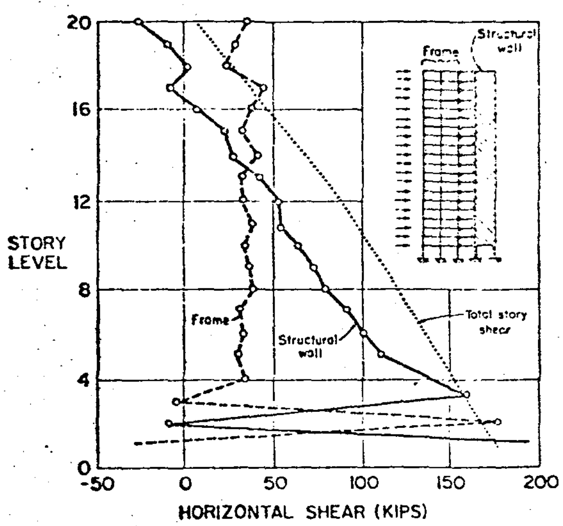

Fig. 2 Distribution of Horizontal Story Shears Between Wall and Frame Under Statically Applied Lateral Load

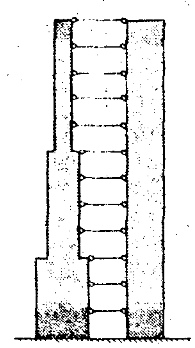

Fig. 3 Linked Structural Walls with Different Stiffness Distribution Along Height

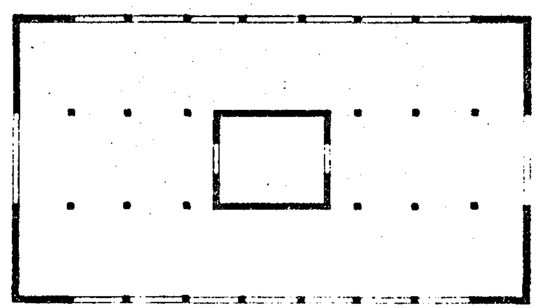

Fig. 4 Example of a Frame-Wall Plan

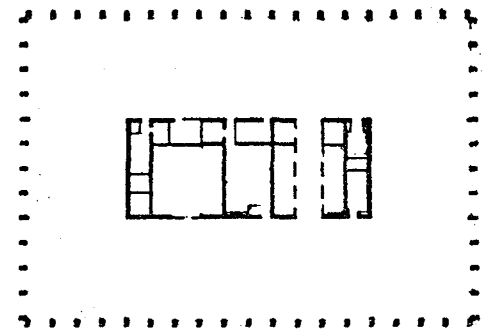

Fig. 5 Typical Tube-in-Tube Plan

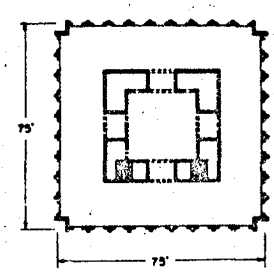

Fig. 6 Banco de America, Nicaragua

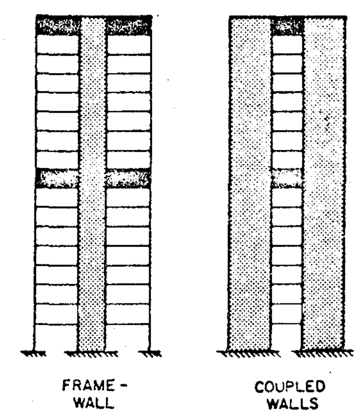

Fig. 7 "Belted" Systems

(Fig. 5). In this system the corewalls are usually perforated for elevator doorways and other openings and thus function as coupled walls in one or both directions.

A more effective disposition of the stiffening walls, particularly with respect to torsional resistance, would have narrower walls located closer to the plan periphery. The walls can then be coupled by beams to increase the overall stiffness of the system and provide the desirable energy-dissipating mechanism in the event of a strong earthquake. The Banco de America building in Managua is a good example of this arrangement (Fig. 6). For certain plan proportions and building uses, however, this type of layout may not be too welcome from the architectural point of view nor too efficient from the mechanical/electrical services standpoint.

Belt Courses

A device used to enhance the coupling between the different vertical elements, and hence increase the overall lateral stiffness, of relatively tall structures is the so-called 'belt course'. This is a one- or two-story-deep beam extending across the width of the structure (Fig. 7). The principal purpose of such heavy beams is to allow the resulting structure to resist the overturning moment due to lateral loads more by cantilever action, that is, by mobilizing to a greater degree the axial resistance of the connected vertical elements. Belt courses are usually located at the top of the structure and at one or more intermediate floors where mechanical equipment and other services can be placed. Studies on the optimal location of belt courses are reported in References 7 and 8.

The use of reinforced concrete belt courses has proved quite effective in systems subjected primarily to wind loading. Their use in earthquake-resistant frame-wall systems, however, may require special attention. The fact that they represent regions of discontinuity along the height of the structure with accompanying high shears, and their appreciably greater stiffness and strength relative to the connected frame columns will almost ensure significant yielding in the columns. Also, the tensile forces that may be developed in the connected vertical elements will tend to reduce their shear (and deformation) capacity. Because of these considerations, the increase in overall lateral stiffness obtained by greater coupling may be outweighed by the negative effects that the use of belt courses may have on the behavior of frame elements attached to it.

Earthquake-Resistant Design Concepts

In addition to the general layout of the building in plan and elevation, there are considerations relating to the manner in which each structural element making up the system is to function under progressively increasing amplitudes of deformation associated with response to strong ground motion.

A major advantage of frame-wall systems, as compared to isolated walls, i.e., parallel walls not in the same plane and connected by floor slabs, is their structural redundancy. This redundancy allows the engineer the option of designing into a structure a hierarchy of elements such that inelastic action occurs first in secondary elements and progresses up the scale to primary elements as the overall deformation of the structure increases. The term secondary is used here to denote elements which are not critical to the overall stability of the structure, i.e., elements in which distress caused by excessive inelastic action cannot seriously undermine the overall gravity-load-carrying capacity of the structure. Secondary elements will generally take the form of beams but may also be vertical walls specifically designed for this purpose. The desirable condition would be to have most of the inelastic action and energy dissipation occur in secondary elements.

Thus, coupled wall systems can be so proportioned that significant yielding under strong ground motion occurs in the coupling beams before inelastic action takes place at the bases of the walls. The so-called "strong column-weak beam" concept used in proportioning moment-resisting frames also serves the same purpose of forcing most of the inelastic action to take place in elements that are less critical to the overall stability of the system. The same general concept applies to frame-wall structures where the wall is designed to be the principal lateral-load-resisting element while the frame carries most of the gravity loads. Figure 8 shows an example of a plan where the walls need not be relied on to carry the gravity loads.

Structural Wall and Frame -- In a frame-wall system consisting of a single (i.e., not coupled) wall connected to a frame, yielding under strong ground motion is most likely to occur first at the base of the wall (unless the beams connecting the wall to the frame are very stiff, i.e., deep or have short spans). In many cases, the wall serves not only as the major lateral-load-resisting element but also carries a significant portion of the gravity loads. The axial compressive forces produced by gravity loads on the wall tend to increase the shear capacity of the wall and help reduce tensile stresses at the foundation level. However, because yielding can occur early at the base of the wall, it is important to design the wall so that its vertical-load-carrying capacity is not impaired as a result of hinging at the base.

Coupled Walls and Frame -- A preferable configuration, and one that occurs often in practice, is a system consisting of coupled walls connected to a frame. In such a system, most of the inelastic action (energy dissipation) can be made to occur in the coupling beams before yielding occurs at the bases of the walls [9,10]. The strength (i.e., yield level) of the coupling beams can be varied along the height of the structure to permit most of these to yield at a predetermined deflection, if desired. Because of the feasibility of controlling the hinging sequence and the relative ease and economy with which the coupling beams in a coupled wall system can be repaired, this type of structure stands out as a most appropriate subsystem for earthquake-resistant reinforced concrete structures. Its superior behavior is such that, even when a solid wall is called for, it may be desirable to deliberately design and detail the wall as a coupled wall system, with nonstructural filler panels used to cover the spaces between coupling beams. This may require compensating for the stiffness lost by introducing coupling beams in place of the solid web of the wall. However, it is believed that this is a reasonable price to pay for an improved performance which allows most of the inelastic action to take place in secondary, easily repairable, elements (i.e., coupling beams) rather than at the critical section near the base of the wall.

In all of the above schemes, it is assumed that the individual structural elements making up the frame-wall system will be designed to provide the necessary strength and deformation capacity. The results of recent tests conducted in various laboratories on large-size coupled wall [4, 11], and isolated wall specimens [12-14], as well as coupling beams [15-18], beam- and

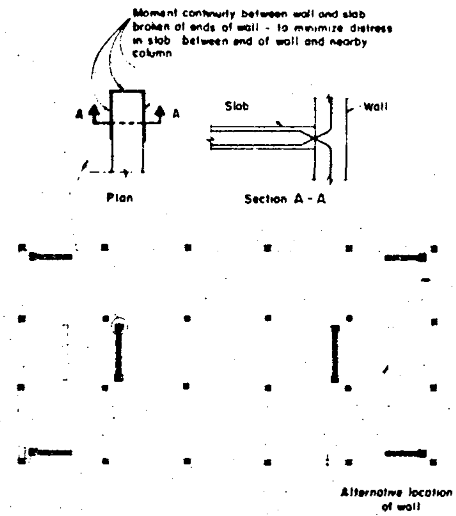

Fig. 8 Walls Functioning Primarily as Lateral-Load-Resisting Elements

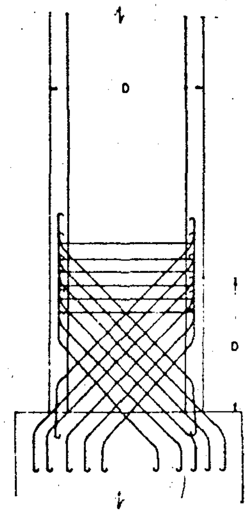

Fig. 9 Diagonal Web Reinforcement at the Base of a Wall

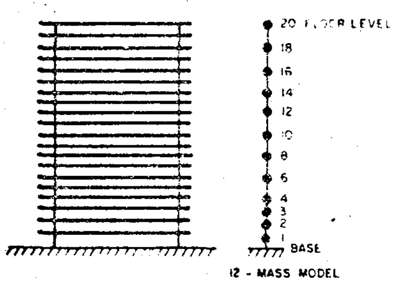

Fig. 10 Isolated Structural Wall Model

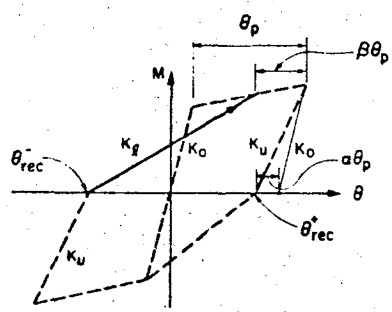

Fig. 11 Decreasing Stiffness Model Parameters α and β

slab-column connections [20-30], and columns [31-33], subjected to slowly reversing loads indicate that it is economically feasible to design frame-wall structures with capacities equal to or greater than the expected demands associated with response to strong ground motions.

Special Details -- In planning a structure for earthquake resistance, certain features (e.g., symmetry, avoidance of major discontinuities, etc.) have been pointed out as being desirable in order to reduce the forces induced in the structure. This objective of reducing the forces induced by ground motions can logically be pursued further by introducing special devices or mechanisms into the structure.

Among a number of proposals advanced to improve the response characteristics of a structure through the use of special devices or mechanisms are: (a) isolation devices to limit the magnitude of the earthquake forces transmitted to the superstructure. These may take the form of ball-bearing or Teflon pads combined with dampers [34, 35] or columns at the base designed to yield at a predetermined deflection, i.e., soft story concept [36, 37]; rocking ball mechanisms [38] and similar devices, and (b) mechanisms designed to provide additional energy dissipation through yielding, either in bending [39] or direct tension [40] of specially mounted steel rods. A method of increasing the lateral deformation capacity of walls by introducing slits into it (i.e., the "slitted wall"), thereby converting it essentially into a series of closely spaced columns has also been used [41].

While these special devices, if reliably designed and properly maintained, can provide some attenuation in response and in a sense increase the margin of safety in design, the basic problem should still be recognized as that of determining reliable estimates of the demands, corresponding to any particular configuration, and the correlation of these with available capacities. Obviously, where conventional systems can be shown to provide the necessary capacity economically, these would be preferred.

THE DESIGN PROBLEM

Basic Requirements

As in all structures to be designed for earthquake resistance, the basic design requirements for frame-wall structures consist of:

(1) Estimates of the force and deformation demands in critical regions of structures corresponding to different combinations of the significant structural and ground motion parameters. These data on demands deal primarily with the requirement of life safety, i.e., the prevention of collapse under the design earthquake.

An auxiliary consideration is the combination of stiffness and strength needed to minimize damage to both structural and nonstructural components by limiting the overall structural displacements.

(2) Estimates of the strength and deformation capacity of typical structural elements corresponding to different values of the significant design parameters, i.e., element cross-section, reinforcement details, axial load, level of shear, etc.

The compilation of comprehensive data on force and deformation demands which can serve as bases for a design procedure will require extensive dynamic inelastic analyses of realistic models of the basic structure. The desired information should show the variation of demand with the significant structural and ground motion parameters. In a similar manner, design data which can be used for proportioning members must be obtained through a systematic test program using large-size specimens subjected to realistic loading conditions.

At present, there is a lack of design information relating to frame-wall systems reflecting a correlation between force and deformation demands with corresponding capacities. There has been no systematic compilation and correlation of data aimed specifically at developing design information for use in everyday practice.

Typical Design Approach

Apart from a straightforward adherence to standard code requirements, the usual approach to the design of multistory buildings which justify a more-than-usual investigation, i.e., beyond that normally required by codes, consists in carrying out elastic time-history analyses of appropriate models using a few input accelerograms [42,43] . Engineering judgment is then used to arrive at design values by allowing for inelasticity developing in critically stressed members on the basis of the calculated elastic forces and displacements.

In other cases, estimates of the maximum overall displacements and the associated forces are obtained by modal superposition using smoothed or averaged response spectra. The most common practice is to take the square root of the sum of the squares [44-46] of the response corresponding to the first few significant modes (assuming the modal frequencies to be spaced far enough from each other). Where the calculated elastic moment is greater than the known yield moment of a member, the ductility requirement is sometimes estimated on the basis of the overstress ratio, i.e., the ratio of the maximum elastic moment to the yield moment.

Comparisons [9, 47] of the results of linear and nonlinear dynamic analyses, however, have shown that while an elastic analysis may provide fair estimates of the maximum overall structural displacements, it can grossly underestimate the magnitude of the inelastic deformations in critical regions of structures. In order to obtain reliable data on deformation demands, inelastic time-history analyses of realistic models are required.

A design procedure proposed by Shibata and Sozen [48] replaces the planar model of a structure by a "substitute (elastic) structure" with reduced stiffness and an equivalent viscous damping based on assumed tolerable damage levels. The design forces are then obtained by a modal superposition analysis of the substitute structure using linear response spectra. The procedure allows, in an approximate and indirect way, for the local concentration of inelastic deformations in critical members.

A more elaborate analysis and design procedure employing both linear and nonlinear time history response analyses was discussed by Bertero and Kamil in Reference 49. The approach includes a logical progression from linear dynamic analysis for preliminary proportioning of elements to a verification of the final design by inelastic dynamic analysis, a procedure clearly desirable for major projects. The method allows an examination of the deformations in critical regions of structures. However, because the procedure requires the use of dynamic analysis programs, its use on moderate-sized projects by the average engineer - who may not have access to the necessary computing facility or even the time to familiarize himself with the programs - may be limited.

The Need for a Simple Rational Design Procedure -- From the point of view of broad application, it would be desirable to have at the disposal of the average engineer relatively simple and practical design information which provides reliable estimates of the force and deformation demands in critical regions of structures as well as guides on the proper proportioning of elements to provide the required capacity. Such information should cover the practical range of variation of the significant design parameters. The development of this design information will obviously require a comprehensive and integrated analytical and experimental program of investigation.

PROPOSED PROCEDURE FOR DEVELOPMENT OF DESIGN INFORMATION

Generation of Data on Demand Through Dynamic Inelastic Analysis

It has often been noted that although our structural analytical capabilities have advanced considerably during the last two decades - mainly as a result of the electronic digital computer - this advance has not been matched by a corresponding improvement in the overall bases for the design of structures, particularly with respect to strong ground motions. While this condition may be typical of scientific progress in general, that is, of theoretical analysis spearheading the development of rational design procedures, it would seem desirable at this point to seek to narrow this gap by taking full advantage of our vastly improved analytical capability to further the aims of structural design.

It is worth noting that except for the investigation of individual structures, most analytical studies on dynamic earthquake response have been concerned mainly with either examining the validity of certain proposed mathematical models of structures or with parametric studies of response. Relatively little effort has been spent in a systematic compilation of force and deformation demands corresponding to different combinations of the significant design variables.

There is no doubt that the development of adequate mathematical models constitutes the first step in the preparation of the necessary tools for dynamic analysis. In assessing the validity of a proposed model - developed to account for an action or mechanism judged to be significant in a structure - the results of dynamic analyses using the model are usually compared with data obtained from shaking table tests of specimens designed specifically for this purpose [10, 50-52], or with observed damage of actual structures subjected to earthquake motions [9,47,53]. The accuracy of the analytical prediction (and hence the validity of the proposed mathematical model) with respect to the observed experimental behavior is generally determined by a comparison of time-history response curves for nodal displacements. In the case of structures damaged by earthquakes, the damage which may be inferred from calculated deformations is compared with the extent of observed damage. In this connection, the importance of using the proper criteria in establishing the equivalence between analytical and experimental results should not be overlooked. For instance, from the design standpoint, it is more important to have reliable estimates of the critical force and deformation demands in local regions of primary elements than of overall or gross structural displacements, which are generally not as sensitive to parameter variations. Thus, agreement between analytical and experimental results in terms of rotational ductilities in critical regions, rather than in terms of overall or top displacements, may be the more significant criterion in such comparisons.

In developing information for use in design practice, a slightly different approach must be taken to utilize our dynamic inelastic analysis capabilities. In contrast to the basic use of analysis to assess the validity of certain mathematical models or modelling techniques, the estimation of critical force and deformation demands in primary elements of typical structural configurations requires the systematic compilation of response data for practical ranges of values of the significant parameters. It is in the generation of comprehensive data on demand to serve as bases for a design procedure that dynamic analysis can find one of its most useful applications. This, of course, assumes the use of an adequate mathematical model as a basic analytical tool. While there will always be room for improving our models, particularly as our knowledge of structural behavior improves, it is believed that we at present have the necessary tools to determine reasonably good estimates of earthquake demands in structures.

Because of the need for simplicity in the design procedure, only the most significant parameters can be considered in the formulation of the design methodology. Therefore, a parametric study to determine the relative importance of the different variables affecting dynamic structural response is necessary.

For the particular case of frame-wall structures, the relative influence of the following basic structural parameters on the force and deformation requirements in critical regions may have to be examined:

1. fundamental period of structure (as affected primarily by stiffness)
2. yield level in flexure of walls
3. yield level in shear of walls
4. coupling beam-to-wall stiffness ratio } where coupled
5. coupling beam-to-wall strength ratio } walls are used
6. frame-to-wall stiffness ratio } 'wall' may be single wall(s) or
7. frame-to-wall strength ratio } coupled walls
8. foundation rocking

Once the significant variables have been isolated, a comprehensive series of analyses can be undertaken to compile data on estimated demands corresponding to measures of available capacity obtainable from experiments. The generation of design data will involve analyses using several input accelerograms of reasonable duration and having frequency characteristics designed to excite a structure critically [54]. Furthermore, analyses using input motions of varying intensity will have to be carried out to obtain data corresponding to varying ranges of expected ground motion intensity. In order for the dynamic response data to fulfill the requirements implied in this application, the analyses must obviously be quite comprehensive.

Development of Experimental Data on Capacity

It is clear that any advance in design capability will have to rely heavily on experimental data concerning behavior of elements and structures, in addition to analytical results. Until relatively recently, little in the way of experimental data has been generated relating to the behavior of typical structural configurations, and particularly of structural walls and wall systems, subjected to earthquake or earthquake-type loading. Whether this is a reflection of funding priorities - given the generally greater cost of experimental programs - or an indication of the preference on the part of many researchers to undertake analytical studies rather than experiments, is not clear. However, it is clear that if a significant advance is to be accomplished in the area of design, a systematic approach combining both analysis and experiment must be considered. Such an effort must be specifically aimed toward the development of design procedures covering the more important structural types.

The development of design data to guide the proportioning and detailing of structural elements and systems for a specified strength and deformation capacity will require the systematic determination of the effects of different structural and loading variables through testing of large-size specimens under representative loading conditions. Such tests, designed to isolate, to the extent possible, the effect of each major variable, would obviously have to be fairly extensive.

Experimental investigation of the effects of the following variables on the strength and deformation capacity of structural walls and wall systems is needed:

1. wall cross-section
2. concrete strength
3. confinement reinforcement
4. shear reinforcement
5. level of applied shear
6. moment-to-shear ratio
7. axial load.

In the process of obtaining experimental information on capacity for different element types, the feasibility of utilizing special reinforcement details not normally used in conventional reinforced concrete construction should be explored [4,11,15,16,19,27] . The effectiveness of alternative details designed to enhance the resistance of elements to cyclic inelastic deformations under high shears should be examined. For instance, the effectiveness of diagonal web reinforcement at the base of structural walls deserves consideration (Fig. 9). It is believed that with proper detailing of the anchorage of the diagonal bars and confinement of the region in the web near the corners of the base, such a detail would prove more effective than horizontal bars. Construction of such a detail need not cause undue difficulties if it is prefabricated and used only in potential hinging regions, especially at the bases of walls.

Correlation of Data on Demand and Capacity

The response data from dynamic inelastic analyses should provide estimates of the stiffness requirements to limit distortions in structures to tolerable levels as well as force and deformation demands corresponding to particular combinations of the significant structural and ground motion parameters. The force and deformation demands in regions of elements which become inelastic are of particular interest, since design attention will have to be focused on these inelastic regions. Such analytically derived data on demand, when correlated with experimental data on capacity, can serve as bases for determining appropriate force levels to be used in design.

The development of practical design information based on a correlation of analytically determined demands and experimentally derived capacities can be tedious but otherwise fairly straightforward. However, relatively little has been done to generate the necessary information and carry out the correlation to the point where results useful to the design engineer can be formulated. An effort along the lines suggested here has been initiated and is now in progress at the Portland Cement Association, for the particular case of isolated structural walls [55]. The project is sponsored in major part by the National Science Foundation. An indication of what can be done for the case of frame-wall structures may be obtained by considering a few of the results of this particular study.

Determination of Design Force Levels (for Isolated Structural Walls)

Figures 12 to 16 illustrate the results of the dynamic analyses of 20-story isolated structural walls (Fig. 10) subjected to input motions having a spectrum intensity* equal to 1.5 times the spectrum intensity of the N-S component of the 1940 El Centro record (= SI_{ref}). The graphs shown in these figures represent the maximum response to six different input motions. Similar graphs have been prepared for walls of different heights and input motion intensities equal to 0.75 and 1.0 (SI_{ref}). The intent in determining the critical dynamic response quantities for different input motion intensities was to have such values available in anticipation of the development of seismic regionalization maps defining zones in terms of the maximum spectrum intensities of the expected motions - or some quanitity related to these - and their corresponding return periods or recurrence intervals.

The structural models used in obtaining Figs. 12 to 16 had the following common characteristics: viscous damping coefficient for first and second modes = 0.05; yield stiffness ratio, i.e., the ratio of the slope of the second, post-yield branch to the slope of the initial 'elastic' branch of the bilinear M-θ curve, = 0.05; parameters characterizing the 'decreasing stiffness' hysteretic loop (Fig. 11): unloading parameter, α = 0.10, reloading parameter, β = 0; stiffness of wall uniform throughout height; strength, i.e., M_y, uniform throughout height except for adjustments to reflect effect of axial load; and, wall fully fixed at base, with the input motion applied directly to base.

Figures 12 and 13 show the variation of the maximum top displacement and interstory displacement, respectively, with the initial fundamental period for different values of the available ductility, μ_a. The essentially identical maximum displacements of structures having different available ductilities (for the same period), a behavior observed earlier with respect to single-

*defined as the area under the 5%-damped velocity response spectrum corresponding to 10 seconds of the ground motion, between periods 0.1 sec to 3.0 sec.

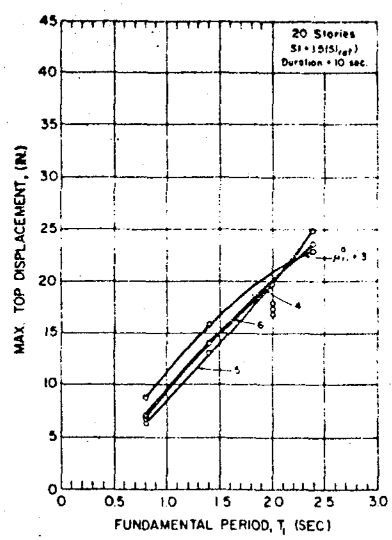

Fig. 12 Max. Top Displacement as a Function of Fundamental Period and Available Ductility - Isolated Structural Walls

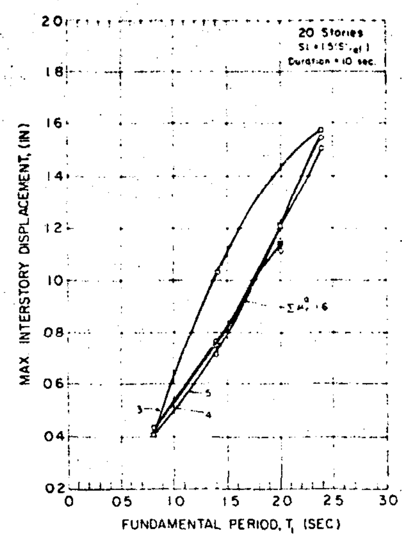

Fig. 13 Max. Interstory Displacement as a Function of Fundamental Period and Available Ductility - Isolated Structural Walls

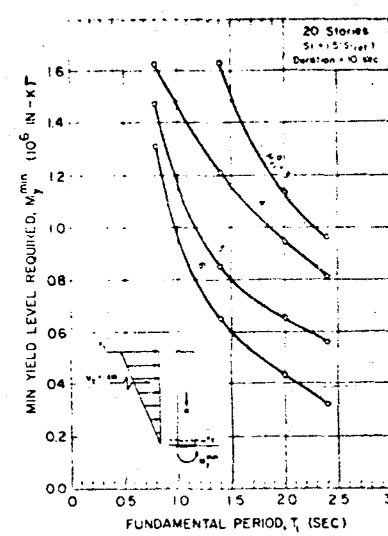

Fig. 14 Min. Yield Level Required at Base of Wall as a Function of Fundamental Period and Available Ductility - Isolated Structural Wall.

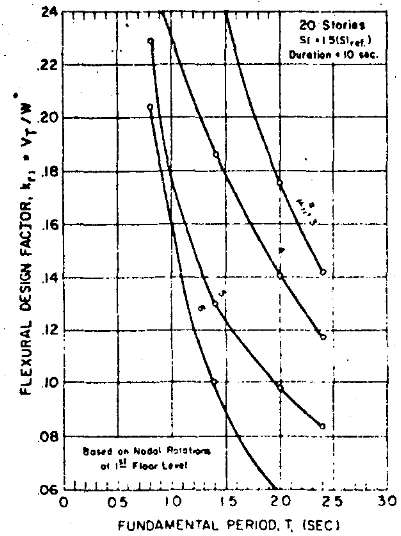

Fig. 15 Flexural Design Factor, k, as a Function of Fundamental Period and Available Ductility - Isolated Structural Walls

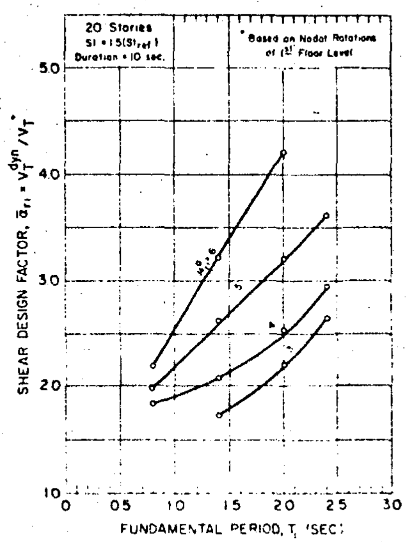

Fig. 16 Shear Design Factor, $\overline{\alpha}$, as a Function of Fundamental Period and Available Ductility - Isolated Structural Walls

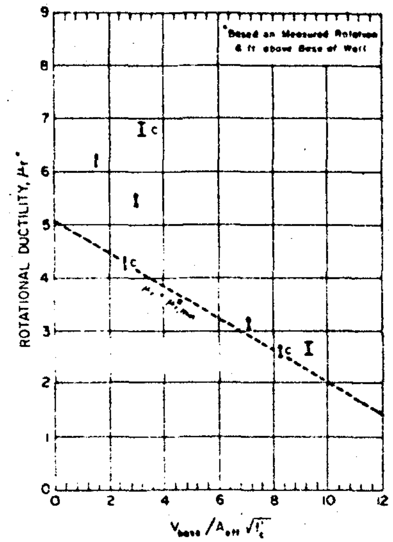

Fig. 17 Measured Available Rotational Ductility, μ_r^a, as a Function of the Applied Maximum Shear Stress - Isolated Structural Walls [see Ref. 14]

degree-of-freedom systems [56] as well as frames [47], will be noted in these figures. The variation of the minimum yield level, M_y, required at the base of the wall with the fundamental period, for different values of the available ductility, is shown in Fig. 14. As might be expected, the figure shows that for a given fundamental period, a higher available ductility implies a lower minimum required strength (yield level) at the base.

In determining ductility requirements for use in design, a study was conducted [55] to assess the relative magnitudes of the different measures of deformation in the hinging region that have been used in the literature. The measures of deformation considered are shown in Fig. 18. These include three measures of rotation and one of rotational energy. The study involved the calculation of all four measures for both dynamic analysis results and test specimens. A comparison of these different measures of deformation indicated that, at least for the samples considered, the satisfaction of the deformation requirement in terms of rotational ductility, μ_r, generally ensures the satisfaction of the other measures of deformation.

Figures 15 and 16, based on dynamic analysis results, show examples of charts that can be used in actual design work. These charts, together with Fig. 17, which summarizes the essential results of tests of isolated structural walls of varying cross-section and detail [14], form the basis for establishing the design force levels to be used in proportioning the structure. The use of the charts is best explained by describing the steps in the design procedure. A similar general procedure can be applied to frame-wall systems, with appropriate modifications to cover the additional considerations involved in the more complex systems.

(1) Preliminary Design -- A logical first step is a design satisfying gravity and wind loading requirements. Here the proper disposition of stiffening elements in plan, with particular regard to symmetry and torsional resistance, cannot be over-emphasized.

From the preliminary design an initial effective stiffness can be assumed and the corresponding initial fundamental period, T_1, determined.

(2) Stiffness Design for Damage Control -- As far as stiffness and the associated displacements due to ground motion are concerned, the major design considerations are (a) the stability of the structure, and (b) damage control. Generally, the considerations related to damage control govern, i.e., the damage control criteria are more stringent than those related to stability.

The maximum tolerable deformation, whether expressed in terms of the ratio of the maximum top displacement to the total height or of the maximum interstory displacement to the story height, which can be considered acceptable in order to limit damage to nonstructural components of buildings has not been clearly defined. Obviously, this will depend on the material of which the critical component is made and the mounting or attachment details used.

Figures 12 and 13, or similar ones for other structure heights and earthquake intensities can be used as guides in selecting the appropriate fundamental period, and hence stiffness, once the tolerable maximum displacement has been selected or assumed.

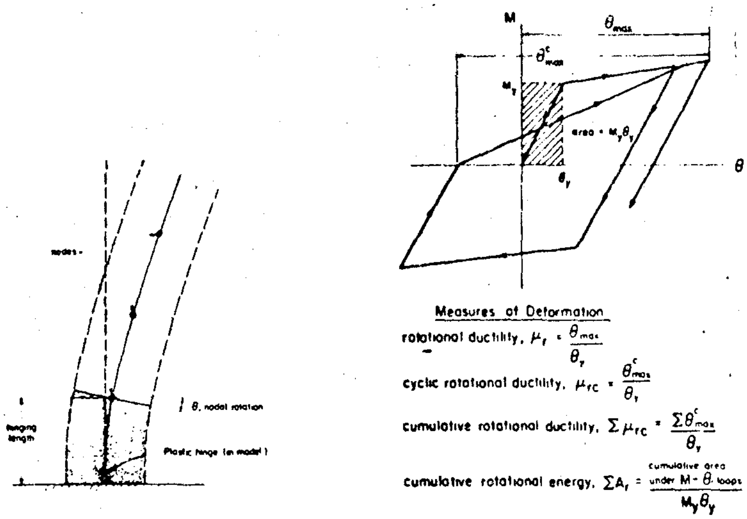

Fig. 18 Different Measures of Flexural Deformation in Hinging Region of Walls Considered in Reference 55

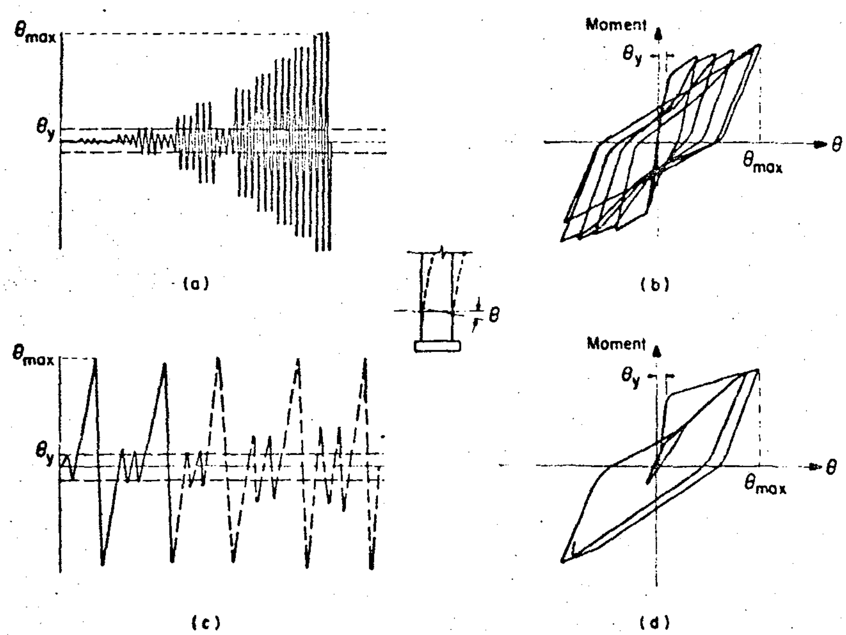

Fig. 19 Sequence of Deformation in Experimental Loading Program
(a) and (b) - Most Commonly Used Program
(c) and (d) - Program Derived on Basis of Study Reported in Ref. 55

(3) <u>Design for Strength and Deformation Capacity: Base of Wall</u> --

(a) Assume an available rotational ductility at the base of the wall, μ_r^a. A trial value may be obtained from a chart such as shown in Fig. 17 (based on experimental data) showing available rotational ductility as a function of the maximum nominal shear stress, by entering the chart with an assumed value of the maximum shear stress.

(b) Determine the minimum yield level required at the base, M_y^{min}, using a chart such as is shown in Fig. 16, giving M_y^{min} as a function of the fundamental period, T_1, and the available ductility, μ_r^a. This value of M_y^{min} can be used to determine the required flexural reinforcement at the base of the wall, if the value of the nominal shear stress assumed in (a) is verified as correct or acceptable.

Also determine the flexural design factor, k, from chart such as is shown in Fig. 15, giving this factor as a function of T_1 and μ_r^a. Then calculate the total horizontal design force, V_T = kW, where W is the total effective weight of the structure.

(c) Determine the shear design factor, $\bar{\alpha}$, from a chart such as Fig. 18, showing this factor as a function of T_1 and μ_r^a. Then calculate the effective static design shear for proportioning the shear reinforcement at the base, $V_{design} = r_v \bar{\alpha} kW = r_v \bar{\alpha} V_T$, where r_v is an appropriate reduction factor intended to account for the over-conservatism inherent in the critical dynamic shears shown in Fig. 16 when compared to the shear capacity obtained from the experimental program.*

(d) Using the experimentally derived chart shown in Fig. 17, or a similar chart, check if the available ductility, μ_r^a, assumed in Step (a) can be developed under the design shear stress determined in Step (c).

If the assumed ductility can be developed, then determine the required shear reinforcement - using design and detailing recommendations developed on the basis of the experimental investigation. If the assumed ductility cannot be developed under the calculated design shear stress, adjust the assumed available ductility value, μ_r^a, and/or modify the wall section dimensions to reduce the shear stress (a recalculation of the period, T_1, may be required in the latter case), and repeat Steps (a) through (d) until a reasonable agreement between assumed and developable ductility is obtained.

The above comparison between assumed and developable values can alternatively be carried out in terms of the shear stress instead of ductility, which can then be assumed as fixed.

(4) <u>Design of Upper Portions of Wall</u> -- Determine flexural and shear reinforcement required in upper portions of wall on the basis of the distribution of V_T (= kW). (The appropriate distribution of V_T is still being studied.)

A check on the ductility requirements in upper portions of the wall, in a manner similar to that used for the base of the wall, may have to be considered.

*The reasons for this over-conservatism are given in Reference 55.

A major distinction between the above-described procedure and current simplified design procedures is the explicit relationship established between the principal structural parameters, i.e., the fundamental period and yield level, and the force and deformation requirements in the critical regions of walls as well as the manner in which these have been correlated with experimental data to yield design forces. A design procedure for frame-wall systems can be developed along similar lines, with appropriate modifications to reflect the effect of other structural parameters characterizing the more complex systems.

<u>Some Questions Concerning Loading</u>

In correlating capacity values obtained from experiments with demands estimated from dynamic inelastic analyses, it is essential that the capacity values be derived under conditions closely approximating those prevailing under dynamic conditions. This is particularly important for those conditions or factors which have significant influence on the behavior of reinforced concrete elements. The validity of any correlation between demand and capacity will depend on how representative the loading conditions used in the laboratory are of actual dynamic response. While there are many aspects to this problem [57], only two factors will be discussed here.

<u>Representative Loading History; Effect of Sequence of Deformation</u> -- For the purpose of obtaining detailed data on specimen behavior for design applications, the most common loading program used in quasi-static tests of large-size specimens under cyclic reversed loads consists of imposing deformation cycles of progressively increasing amplitudes until failure occurs [11-33] (Fig. 19(a) and (b)). The maximum forces and deformations sustained are then noted as indicating capacity. It has been suggested by Bertero [57] that such a loading program may not be as conservative as a program in which the peak deformation is imposed early in the test.

The development of a 'representative' loading history for critical regions in structures which can be used in testing large-size specimens under slowly applied reversing loads is one of the more important results that can be obtained from dynamic inelastic analyses. Such a loading program would have to be defined in terms of the maximum amplitude of deformation, the number of cycles of large amplitude and the sequence in which the large-amplitude cycles occur, with particular reference to the first large-amplitude cycle. The deformation of interest in most cases will be the total rotation that occurs in the hinging region of elements. In addition, the intensity of the accompanying shear and axial load and the variation of these relative to the deformation will have to be noted.

In application, laboratory tests using a loading history developed on the basis of dynamic analyses will have the character of proof tests. A specimen that sustains such a loading program without significant loss of strength can be said to be adequate with respect to design and details for the particular combination or range of values of the significant design variables represented by the loading program.

A study of loading history for isolated structural walls now underway at PCA, for example, indicates that the maximum number of large-amplitude (i.e., 0.75-1.0 of the maximum) cycles of deformation at the critical section near the base rarely exceeds six for a 20-second duration input motion [58]. The input accelerograms used were synthesized by repeating the first ten seconds of strong motion to give a total of twenty seconds. Samples of the composite 20-second accelerograms used in the study are shown in Fig. 20.

Figure 21 shows histograms indicating the number of "fully reversed cycles" and the number of inelastic cycles of rotational deformation calculated at the base of the wall. For the purpose of Fig. 21(a), a "fully reversed" cycle was defined as a cycle with at least one peak value between 0.75 and 1.0 of the calculated maximum amplitude and the other - on reversal - between 0.50 and 1.0 of the maximum. A total of 170 cases are represented in Fig. 21, covering wall heights from 10 to 40 stories, fundamental period values from 0.8 to 3.0 seconds, yield level values ranging from 33,890 to 338,940 kN.m (300,000 to 3,000,000 in-kips) and spectrum intensity values for the input motions from 0.75 to 1.5 times that corresponding to the first 10 seconds of the N-S component of the 1940 El Centro record (Imperial Valley earthquake). A total of 10 different input motions were used, including one artificially generated accelerogram. Further details of the study are reported in Reference 58.

The inelastic cycles plotted in Fig. 21(b) include "large" and "small" amplitude inelastic cycles, a large amplitude being defined as an inelastic rotation between 0.75 and 1.0 of the corresponding maximum. The amplitude of a wave in all cases was measured from the initial (zero) position. Thus, in a rotation-vs.-time plot such as is shown in Fig. 22, a rotation cycle that exceeds the horizontal line representing the initial yield value was considered inelastic.

Of particular interest insofar as sequence of loading is concerned is the fact that in many cases, a deformation equal or close to the maximum occurs quite early in the response, with hardly any inelastic cycle preceding it. This is indicated, for example, in Fig. 22 which shows the history of rotational response of the node at the first floor level (representing the total rotation occurring in the segment between the fixed base and the first floor level) of 20-story walls subjected to the first 10 seconds of the E-W component of the 1940 El Centro record. A plot summarizing the information on this particular aspect of response for the 170 cases studied is shown in Fig. 23. This figure clearly demonstrates that for the particular type of structure considered, it is reasonable to expect a deformation amplitude equal or close to the maximum occurring early in the response, with no inelastic cycle preceding it.

Preliminary results of tests on isolated structural walls conducted at PCA and designed to verify the effect of sequence of loading indicate that a loading program in which the design maximum deformation is imposed early in the test, with only an elastic cycle preceding it, as shown in Fig. 19(c) and (d), can be much more severe than a program consisting of reversed cycles of loading with amplitudes progressively increasing to the maximum (Fig. 19(a)).

Effect of Character of Shear Loading -- In addition to deformation history, it is important, in simulating earthquake response through quasi-static tests, to impose on specimens the forces that analyses indicate may reasonably accompany the maximum deformations. This is particularly important in the case of shear because of its significant influence on behavior.

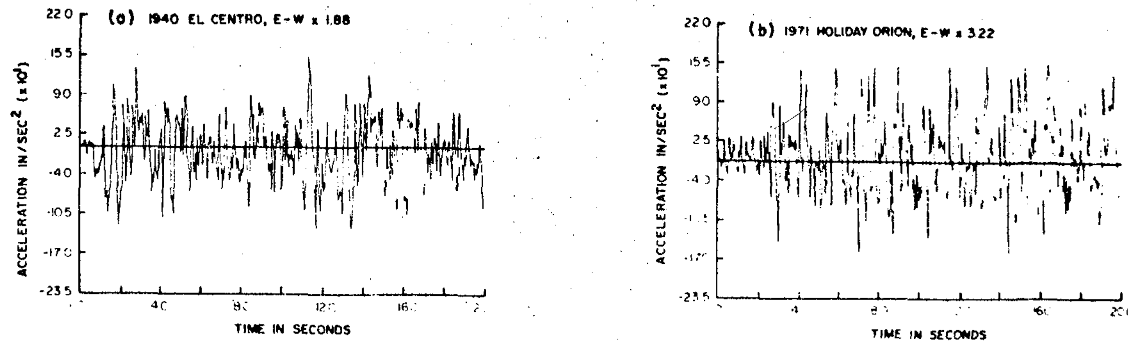

Fig. 20 Composite 20-Second Accelerograms

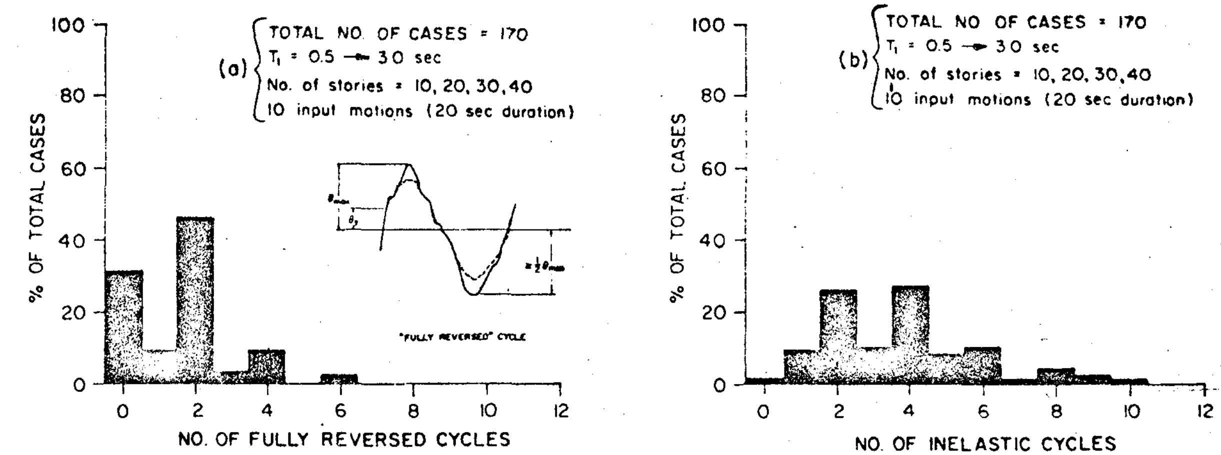

Fig. 21 Histograms Showing Distribution of "No. of Fully Reversed Cycles" and "No. of Inelastic Cycles" - Isolated Structural Walls

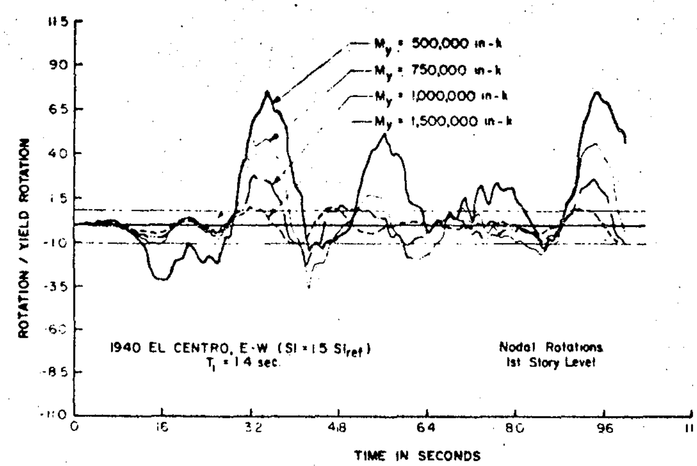

Fig. 22 Variation of Rotation in First Story with Time - 20-Story Isolated Structural Walls

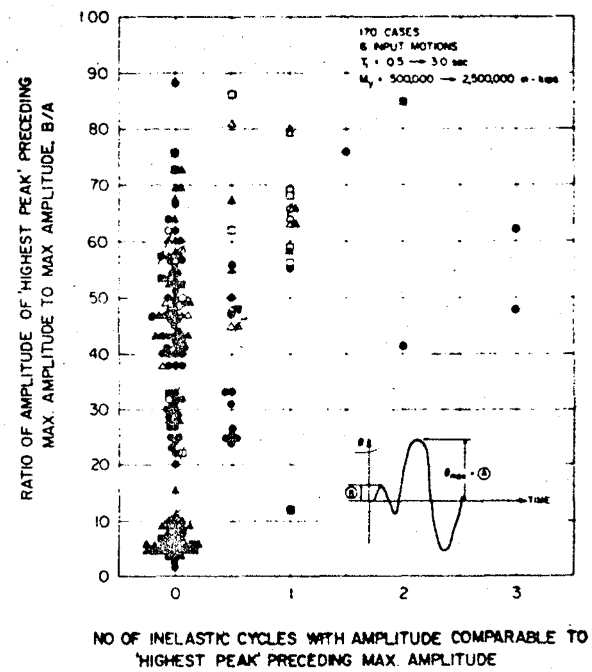

Fig. 23 Number of Inelastic Cycles Preceding Maximum Amplitude Deformation

As far as the shear force used in tests is concerned, two aspects have to be considered. First is the magnitude of the maximum shear force. The second is its variation with time, and particularly in relation to the accompanying moment and deformation. Most of the quasi-static tests that have been conducted to date have been concerned mainly with the magnitude of the expected shear forces. The loading imposed on test specimens [11-33] has been characterized by the moment, shear and the deformation in the critical region being all in phase.

The response studies of isolated structural walls undertaken at PCA[57], however indicate that the shear in the critical region at the base is more sensitive to higher mode response and thus fluctuates more rapidly with time than either the moment or the rotation. This is illustrated in Fig. 24(a) and (b) which show time-history plots of the shear, rotation and moment in the first story of an isolated wall subjected to two different input motions.

The behavior of the shears shown in Fig. 24 may be partly due to the fact that the hinging region in the model used allowed yielding in flexure only, while remaining linearly inelastic with respect to shear throughout the response. Experimental studies [12,14] have shown that this is generally not the case. Whatever the effect of this modelling assumption may be*, it is important in correlating experimental data on capacity with analytical data on demand to allow for possible differences in the manner in which shear is induced under dynamic response conditions and in the typical quasi-static test. It is believed that a shear force that fluctuates rapidly and reaches its peak value only for very short durations relative to the associated moment and rotation does not represent as severe a loading condition as one in which the shear, moment and deformation are all in phase.

SUMMARY

The introduction of reinforced concrete structural walls or coupled walls into frames to form frame-wall structures combines the gravity-load-carrying efficiency of the rigid (open) frame with the lateral-load-resisting efficiency of the structural wall. In planning such structures, a conscious effort can be made to take full advantage of the redundancy in such systems by allowing most of the inelastic action under strong ground motion to take place in elements that are not too critical to the overall stability of the system. By providing sufficient stiffness, strength and deformation capacity in a hierarchy of elements such that a logical sequence of inelastic action occurs under progressively increasing deformations, a reliable energy-dissipative mechanism can be provided while at the same time ensuring the overall integrity of the structure. In this respect, the coupled wall-frame system offers the most effective configuration. The performance of frame-wall structures during recent earthquakes has shown that such a system, when properly conceived and designed, provides an efficient solution to the twin requirements of life safety (i.e., no collapse) and damage control in earthquake-resistant buildings.

*a model which will allow yield in shear, based on uncoupled behavior relative to moment, has been developed to study this and related questions concerning shear yielding.

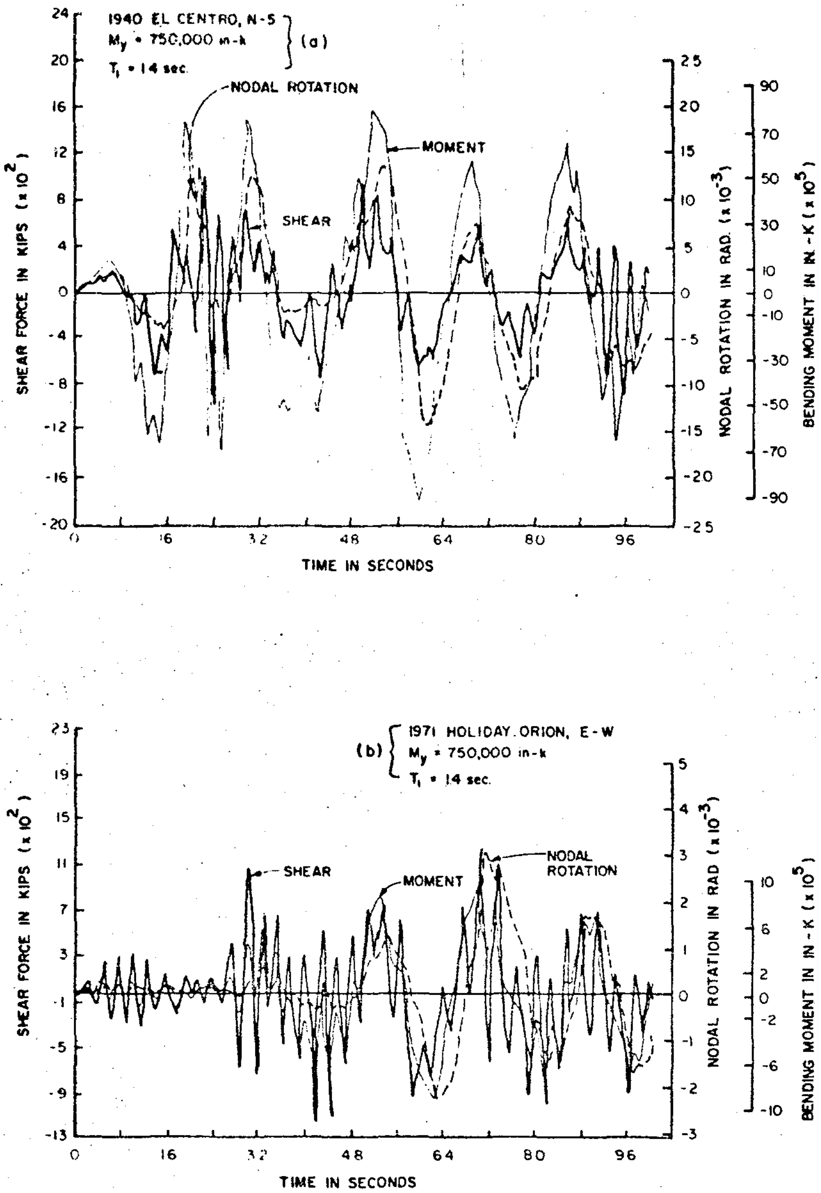

Fig. 24 Variation of Shear and Moment at Base and Rotation in First Story with Time - 20-Story Isolated Structural Walls

The question of sufficiency of design, however, depends primarily on the availability of reliable estimates of demand as well as capacity. At present, there is a lack of information concerning both the force and deformation demands in critical regions of frame-wall structures and the capacity of typical elements (particularly walls) subjected to reversed cycles of loading. A systematic compilation of data on both demand and capacity in frame-wall systems will be required before a practical and reliable design procedure can be developed. Such information should cover a reasonably wide range of values of the major design variables.

A design procedure for earthquake-resistant isolated structural walls has been discussed briefly. It is suggested that a similar approach can be adopted, with appropriate modifications, for the case of frame-wall systems.

Acknowledgment. Work on this report has been supported in part by the National Science Foundation through a Grant (No. GI-43880) for the investigation of "Structural Walls in Earthquake-Resistant Structures."

REFERENCES

1. Fintel, M., "Quake Lesson from Managua: Revise Concrete Building Design?" Civil Engineering, ASCE, August 1973, pp. 60-63.

2. Anonymous, "Managua, If Rebuilt on Same Site, Could Survive Temblor," Engineering News Record, December 6, 1973, p. 16.

3. Sozen, M.A. and Matthiesen, R.B., "Engineering Report on the Managua Earthquake of 23 December 1972," National Academy of Sciences, Washington, D.C., 1975, 111 pp.

4. Paulay, T., "Design Aspects of Shear Walls for Seismic Areas," Research Report 74-11, University of Canterbury, Christchurch, New Zealand, October 1974.

5. Derecho, A.T., "Frames and Frame-Shearwall Systems," Response of Multistory Concrete Structures to Lateral Forces, ACI Publication SP-36, American Concrete Institute, 1973, pp. 13-37.

6. Newmark, N.M., "Torsion in Symmetrical Buildings," Proceedings Fourth World Conference on Earthquake Engineering, Santiago, Chile, Vol. 11, 1969, pp. A3-19 to A3-32.

7. Taranath, B.S., "Optimum Belt Truss Locations for High Rise Structures." The Structural Engineer (London), Vol. 53, No. 8, August 1975, pp. 345-347.

8. McNabb, J.W. and Muvdi, B.B., "Analysis of Belt Trusses on High Rise Structures," Proceedings of the 15th Midwestern Mechanics Conference, Vol. 8, Univ. of Illinois at Chicago Circle, March 1977, pp. 54-56.

9. Mahin, S.A., and Bertero, V.V., "Nonlinear Seismic Response of a Coupled Wall System," Proc. Paper No. 12407, Journal of the Structural Division, ASCE, Vol. 102, ST9, September 1976, pp. 1759-1780.

10. Aristizabal, J.D. and Sozen, M.A., "Behavior of Ten-Story Reinforced Concrete Walls Subjected to Earthquake Motions," Structural Research Series No. 431, Univ. of Illinois at Urbana-Champaign, October 1976.

11. Paulay, T. and Santhakumar, A.R., "Ductile Behavior of Coupled Shear Walls," Proc. Paper No. 11852, Journal of the Structural Division, ASCE, Vol. 102, ST1, January 1976, pp. 93-108.

12. Popov, E.P. and Bertero, V.V., "On Seismic Behavior of Two R/C Structural Systems for Buildings," Paper No. 8, Structural and Geotechnical Mechanics, W.J. Hall, Ed., Prentice-Hall, Inc. 1977.

13. Bertero, V.V., Popov, E.P. and Wang, T., "Seismic Design Implications of Hysteretic Behavior of Reinforced Concrete Elements Under High Shear," U.S.-Japan Hawaii Meeting, August 17-19, 1975.

14. Oesterle, R.G., Fiorato, A.E., Johal, L.S., Carpenter, J.E., Russell, H.G. and Corley, W.G., "Earthquake-Resistant Structural Walls - Tests of Isolated Walls," Report to the National Science Foundation, Portland Cement Association, Skokie, Illinois, November 1976.

15. Paulay, T., "Simulated Seismic Loading of Spandrel Beams," Proc. Paper No. 8365, Journal of the Structural Division, ASCE, Vol. 97, No. ST9, September 1971, pp. 2407-2419.

16. Barney, G.B., Shiu, K.N., Rabbat, B.G. and Fiorato, A.E., "Earthquake-Resistant Structural Walls - Tests of Couplings Beams," Progress Report to the National Science Foundation, Portland Cement Association, Skokie, Illinois, October 1976.

17. Bouwkamp, J.G. and Kustu, O., "Experimental Study of Spandrel Wall Assemblies," Proc. Paper No. 89, Fifth World Conference on Earthquake Engineering, Rome 1973.

18. Higashi, Y., Okubo, M. and Iida, K., "Elasti-Plastic Behavior of Reinforced Concrete Beams with Spandrel Walls under Anti-symmetric Cyclic Loads," Proc. Paper No. 140, Fifth World Conference on Earthquake Engineering, Rome 1973.

19. Bertero, V.V., Popov, E.P. and Wang, T., "Hysteretic Behavior of Reinforced Concrete Flexural Members with Special Web Reinforcement," Report No. EERC 74-9, University of California, Berkeley, August 1974.

20. Popov, E.P., Bertero, V.V. and Krawinkler, H., "Cyclic Bhehavior of Three R.C. Flexural Members with High Shear," Report No. EERC 72-5, University of California, Berkeley, October 1972.

21. Hanson, N.W. and Conner, H.W., "Seismic Resistance of Reinforced Concrete Beam-Column Joints," Proc. Paper 5537, Journal of the Structural Division, ASCE, Vol. 93, No. ST5, October 1967, pp. 553-560.

22. Hanson, N.W., "Seismic Resistance of Concrete Frames with Grade 60 Reinforcement," Proc. Paper 8180, Journal of the Structural Division, ASCE, Vol. 97, No. ST6, June 1971, pp. 1685-1700.

23. Park, R. and Paulay, T., "Behavior of Reinforced Concrete External Beam-Column Joints under Cyclic Loading," Paper No. 88, Proceedings of the Fifth World Conference on Earthquake Engineering, Rome, 1973.

24. Megget, L.M. and Park, R., "Reinforced Concrete Exterior Beam-Column Joints under Seismic Loading," New Zealand Engineering (Wellington), Vol. 26, No. 11, Nov. 15, 1971, pp. 341-353.

25. Renton, G.W., "The Behavior of Reinforced Concrete Beam-Column Joints under Cyclic Loading," M. Eng. Thesis, Univ. of Canterbury, Christchurch, New Zealand, 1972, 163 pp.

26. Uzumeri, S.M. and Seckin, M., "Behavior of Reinforced Concrete Beam-Column Joints Subjected to Slow Load Reversals," Publication No. 74-05, Department of Civil Engineering, University of Toronto, March 1974, 84 pp.

27. Paulay, T. and Spurr, D.D., "Frame-Shear Wall Assemblies Subjected to Simulated Seismic Loading," Proc. Preprints Vol. 3, Sixth World Conference on Earthquake Engineering, New Delhi, India, Jan. 1977, pp. 3-221 to 3-226.

28. De Paiva, H.A.R. and Zaghlool, E.R.F., "Seismic Resistance of Flat Plate Corner Connections," Transactions, Canadian Society for Civil Engineering, Vol. 56, No. 9, September 1973, pp. I-X.

29. Carpenter, J.E., Kaar, P.H. and Corley W.G., "Design of Ductile Flat Plate Structures to Resist Earthquakes," Proc. Paper No. 250, Fifth World Conference on Earthquake Engineering, Rome 1973.

30. Hawkins, N.M., "Slab-Column Connections Subjected to High Intensity Shears and Transferring Reversed Moments," Report SM 76-2, Department of Civil Engineering, University of Washington, Seattle, Washington, 1976, 150 pp.

31. Wight, J.K. and Sozen, M.A., "Shear Strength Decay in Reinforced Concrete Columns Subjected to Large Deflection Reversals," Report No. SRS 403, Department of Civil Engineering, University of Illinois, Urbana, August 1973, 290 pp.

32. Wight, J.K. and Kanoh, Y., "Shear Failure of Reinforced Concrete Columns Subjected to Cyclic Loading," Proc. Paper No. 94, Fifth World Conference on Earthquake Engineering, Rome 1973.

33. Hirosawa, M., Ozaki, M. and Wakabayashi, M., "Experimental Study on Large Models of Reinforced Concrete Columns," Proc. Paper No. 96, Fifth World Conference on Earthquake Engineering, Rome 1973.

34. Anonymous, "Building Will Roll on Ball Bearings During Earthquakes," Engineering News Record, September 24, 1970, p. 14.

35. Capse, M.S., "Earthquake Isolation of Multistory Buildings," Journal of the American Concrete Institute, Vol. 67, No. 11, November 1970, pp. 923-932.

36. Fintel, M. and Khan, F.R., "Shock-Absorbing Soft Story Concept for Earthquake Structures," Journal of the American Concrete Institute, Vol. 66, No. 5, May, 1969, pp. 381-390.

37. Chopra, A.K., Clough, D.P. and Clough, R.W., "Earthquake Resistance of Buildings with a 'Soft' First Story," Earthquake Engineering and Structural Dynamics, Vol. 1, 1973, pp. 347-355.

38. Matsushita, K. and Izumi, M., "Studies on Mechanisms to Decrease Earthquake Forces Applied to Buildings," Proceedings of the Fourth World Conference on Earthquake Engineering, Vol. II, Chile, 1969, pp. B-3-117 to B-3-129.

39. Matsushita, K. and Izumi, M., "Application of Input Controlling Mechanisms to Structural Design of a Tall Building," Paper No. 373, Proceedings of the Fifth World Conference on Earthquake Engineering, Rome, 1973.

40. Anonymous, "An Innovation is Earthquake Resistance," (St. Joseph Hospital and Health Care Center, Tacoma, Washington), Case History Report, Concrete Reinforcing Steel Institute Bulletin 7504.

41. Muto, K., "Earthquake-Resistant Design of 36-Storied Kasumigaseki Building," Special Report, Proceedings of the Fourth World Conference on Earthquake Engineering, Chile, 1969.

42. Housner, G.W., "Earthquake-Resistant Design of High-Rise Buildings," DRC-73-01, California Institute of Technology Earthquake Engineering Research Laboratory, Pasadena, Calif., July 1973.

43. Biggs, J.M., Hansen, R.J. and Holley, Jr., M.J., "On Methods of Structural Analysis and Design for Earthquake," Paper No. 6, Structural and Geotechnical Mechanics, W.J. Hall, Ed., Prentice-Hall, 1977.

44. Merchant, H.C. and Hudson, D.E., "Mode Superposition in Multidegree of Freedom Systems using Earthquake Response Spectrum Data," Bulletin of the Seismological Society of America, Vol. 52, No. 2, April 1962, pp. 405-416.

45. Newmark, N.M., "Current Trends in the Seismic Analysis and Design of High-Rise Structures," Chapter 16 of Earthquake Engineering, R.L. Wiegel, Ed., Prentice-Hall, 1970, pp. 403-424.

46. Housner, G.W., "Design Spectrum," Chapter 5 of Earthquake Engineering, R.L. Wiegel, Ed., Prentice Hall, 1970, pp. 93-106.

47. Clough, R.W. and Benuska, K.L., "FHA Study of Seismic Design Criteria for High-Rise Buildings," HUD TS-3, U.S. Dept. of Housing and Urban Development, Federal Housing Administration, Washington, D.C. 20410, August 1966.

48. Shibata, A. and Sozen, M.A., "Substitute-Structure Method for Seismic Design in R/C," Journal of the Structural Division, ASCE, St1, January 1976, Paper No. 11824, pp. 1-18.

49. Bertero, V.V. and Kamil, H., "Nonlinear Seismic Design of Multistory Frames," Canadian Journal of Civil Engineering, Vol. 2, No. 4, December 1975, pp. 494-516.

50. Otani, S. and Sozen, M.A., "Behavior of Multistory Reinforced Concrete Frames During Earthquakes," Univ. of Illinois at Urbana-Champaign, Structural Research Series No. 392, November 1972.

51. Hidalgo, P. and Clough, R.W., "Earthquake Simulator Study of a Reinforced Concrete Frame," University of California, Berkeley, Report No. EERC 74-13, December 1974.

52. Ackroyd, M.H. and Biggs, J.M., "The Formulation and Experimental Verification of Mathematical Models for Predicting Dynamic Response of Multistory Buildings," Massachusetts Institute of Technology, Cambridge, Structures Publication No. 394, May 1973.

53. Odaka, T., Suzuki, T. and Kinoshita, K., "Nonlinear Response Analysis of Multistory Structures Including Rocking and Swaying Vibration Subjected to Earthquake Motions," Proc. of the Fourth World Conference on Earthquake Engineering, Vol. II, Chile, January 1969, pp. A-4-105 to A-4-121.

54. Derecho, A.T., Ghosh, S.K., Iqbal, M., Freskakis, G.N., Fugelso, L.E. and Fintel, M., "Structural Walls in Earthquake-Resistant Structures - Analytical Investigation, Dynamic Analysis of Isolated Structural Walls," (Part A: Input Motions and Part B: Parametric Studies), Report to the National Science Foundation, RANN, under Grant No. GI-43880, Portland Cement Association, October 1976.

55. Derecho, A.T., Ghosh, S.K., Iqbal, M. and Fintel, M., "Structural Walls in Earthquake-Resistant Structures - Analytical Investigation, Dynamic Analysis of Isolated Structural Walls," (Part C: Development of Design Procedure - Design Force Levels), Interim Report to the National Science Foundation, RANN, under Grant No. GI-43880, Portland Cement Association, November 1976.

56. Blume, J.A., Newmark, N.M. and Corning, L.H., "Design of Multistory Reinforced Concrete Buildings for Earthquake Motions," Portland Cement Association, 1961.

57. Bertero, V.V., "Identification of Research Needs for Improving Aseismic Design of Building Structures," Report No. EERC 75-27, University of California, Berkeley, September 1975.

58. Derecho, A.T., Iqbal, M., Ghosh, S.K. and Fintel, M., "Structural Walls in Earthquake-Resistant Structures - Analytical Investigation, Dynamic Analysis of Isolated Structural Walls, (Part D: Loading History Study), Report to the National Science Foundation, RANN, under Grant No. GI-43880, Portland Cement Associaton, (to be published).

DESIGN OF FRAME WALL STRUCTURES

by

Arnaldo T. Derecho,
Manager, Structural Analytical Services
Portland Cement Association, Skokie, Illinois

DRAFT RECOMMENDATION

1. IF A PRACTICAL AND RELIABLE DESIGN PROCEDURE FOR FRAME-WALL STRUCTURES IS TO BE DEVELOPED, A SYSTEMATIC COMPILATION OF DATA ON FORCE AND DEFORMATION DEMANDS IN POTENTIALLY CRITICAL REGIONS OF TYPICAL STRUCTURAL CONFIGURATIONS AS WELL AS ON THE CAPACITY OF TYPICAL ELEMENTS UNDER REALISTIC LOADING CONDITIONS MUST BE UNDERTAKEN. THE DATA SHOULD COVER A FAIRLY BROAD RANGE OF VALUES OF THE MAJOR VARIABLES.

There is at present a definite lack of both analytical and experimental data upon which to base a correlation leading to a practical and reliable design procedure. Comprehensive data on demand can at present be obtained only through dynamic inelastic analyses of realistic models of structures, while information on capacity, to be convincing, must be derived from experimental tests of large-size specimens subjected to realistic loading conditions. In regard to the latter, questions relating to the sequence of large-amplitude loading cycles and the variation of the applied shear force relative to the moment and rotation need further study before additional tests are undertaken.