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STRUCTURAL SYSTEMS FOR EARTHQUAKE RESISTANT CONCRETE BUILDINGS

by

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Reinforced concrete structural systems that have evolved over the past few decades are reviewed with respect to seismic resistance of buildings of different heights. Lateral load resisting reinforced concrete structural systems are described followed by a consideration of the criteria governing the earthquake resistant design of buildings. Possibilities of controlling ground motion input to building foundations are examined. Requirements of safety or prevention of collapse and damage control, and the design of structures to satisfy these specifications are detailed. Planning and design factors important even at the preliminary design stage are outlined. The seismic performance of reinforced concrete structural systems and the potential prediction of such performance are discussed. Prestressed and precast concrete structural systems are considered. Conclusions drawn are as follows: (1) protection of life and damage control are attainable in buildings stiffened by properly proportioned and detailed structural walls, (2) incompatibility of flexible frames with brittle infills and finishes causes high economic building damage in many earthquakes, and (3) effects of rigid elements on the seismic performance of structures renders it imperative that such elements be accounted for in the design of buildings.

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ABSTRACT

The reinforced concrete structural systems currently used in buildings of various heights are reviewed in this paper. The aspect of primary concern is the seismic resistance of these systems. The criteria governing the earthquake resistant design of buildings are examined. The possibilities of controlling the ground motion input to building foundations are explored. The dual requirements of safety or prevention of collapse and damage control, and the design of structures to satisfy these requirements, are discussed in detail. Some planning and design considerations which are important even at the preliminary design stage are outlined. The seismic performance of reinforced concrete structural systems and the possible prediction of such performance are discussed. A brief consideration of prestressed (including precast) concrete structural systems is included. An enumeration of future research needs concludes the paper. Considerations of foundation design and of soil-structure interaction are beyond the scope of this report.

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INTRODUCTION

The vertical load-resisting capability of a building is its reason for existence. However, with increasing building height, the lateral loads due to wind, earthquakes, etc. assume more and more importance. This is because, with increasing height, the overturning effect of such loads increases. Also, with increasing slenderness, the lateral displacements and interstory displacements may endanger overall structural stability and the integrity of nonstructural elements, and may cause discomfort to occupants. The challenge to the structural engineer in designing a multistory structural system lies in providing the necessary stiffness against lateral loads in a way which will require the least premium for height over the cost of supporting the gravity loads. Structural engineers have met this challenge by developing efficient, economical and innovative new structural systems for buildings ranging in height to over 100 stories.

This paper reviews the reinforced concrete structural systems that have evolved over the last few decades. Resistance to wind was the prime consideration in their development, since, until relatively recently, tall buildings were mostly built in nonseismic areas. This report focuses on the seismic resistance of these structural systems. An important distinction must be drawn here between forces due to wind and those produced by earthquakes. These loads are sometimes thought to belong to the same category, just because codes specify both in terms of equivalent static forces. Although both wind and earthquake loads are dynamic in character, a basic difference exists in the manner in which they are induced in a structure. Whereas wind loads are external loads applied, and hence proportional, to the exposed surface of a structure, earthquake loads are essentially inertial forces. The latter result from the distortion produced by both the earthquake motions and the inertial resistance of the structure. Their magnitude is thus a function of the mass of the structure, rather than its exposed surface. Also, in contrast to structural response to essentially static gravity loading or even to wind loads, which can often be validly treated as static loads, the dynamic character of the response to earthquake excitation can seldom be ignored.

The lateral load resisting reinforced concrete structural systems are described here in general terms, before converging on the seismic resistance of such systems.

LATERAL LOAD RESISTING REINFORCED CONCRETE SYSTEMS

The three basic framing systems to resist lateral loads in high-rise concrete buildings are: (1) frames, (2) structural (shear) walls coupled or acting individually, and (3) frames interacting with structural walls.

Reinforced concrete frame structures depend mainly on the rigidity of member connections for their resistance to lateral forces, and generally tend to be uneconomical (require high premium for height) beyond 20 stories. This is particularly true of the conventional frame structure consisting of two sets of mutually perpendicular frames. Recourse usually has to be taken to other systems for buildings taller than 15 to 20 stories high [1].

The introduction of deep vertical elements or structural walls represents a structurally efficient solution to the problem of stiffening a frame system. This is illustrated in Fig. 1a. A structural wall behaves essentially as a vertical cantilever beam, while a frame exhibits the deformations typical of a shear beam under transverse loads. The interaction between the two elements reduces the lateral deflection of the structural wall at the top, while the wall helps support the frame near the base. However, except perhaps where the walls are located along the exterior of a building or form the elevator shaft, some degree of architectural flexibility may have to be sacrificed with their use. In many cases, a judicious disposition of walls in plan allows them to function efficiently as vertical and lateral load resisting elements without interfering much with architectural requirements. Fig. 1b shows typical plan arrangements of frame-wall systems. Reinforced concrete structures using systems similar to these have been built to a height of 70 stories [1].

Utilization of shear panels--structural walls, one or a few stories in height--scattered throughout the plan, and shifted in location to offer architectural flexibility while supplying sufficient rigidity, is another way to adapt to diverse architectural and functional requirements [2].

The staggered wall-beam system is an innovation suitable for residential buildings. Although only a limited number of high-rise buildings have been built using this system, its advantages of large unobstructed areas in the typical floor, of column-free areas under the buildings for parking, and of high rigidity in the transverse direction may eventually lead to a broader application [2].

A modification of the conventional frame arrangement which has been found economically suitable for buildings up to about 60 stories high is the so-called "framed tube". In this structure, a typical plan of which is shown in Fig. 1c, the exterior columns in what would otherwise be a conventional frame are spaced more closely together and are connected by relatively deep spandrel beams to form an exterior grid which is usually designed to resist the bulk of the lateral load. The framed tube represents a logical evolution of the conventional frame structure, possessing the necessary lateral stiffness with excellent torsional resistance, while retaining the planning flexibility which isolated interior columns allow [1,3].

For taller structures, an arrangement which has been found particularly suitable for office buildings is the so-called "tube-in-tube" system [1,3]. A typical plan is shown in Fig. 1d. This system has emerged as a logical solution to the problem of providing a tall, stiff structure with wide column-free spaces between a central core which houses all services and an external peripheral grid of closely spaced elements.

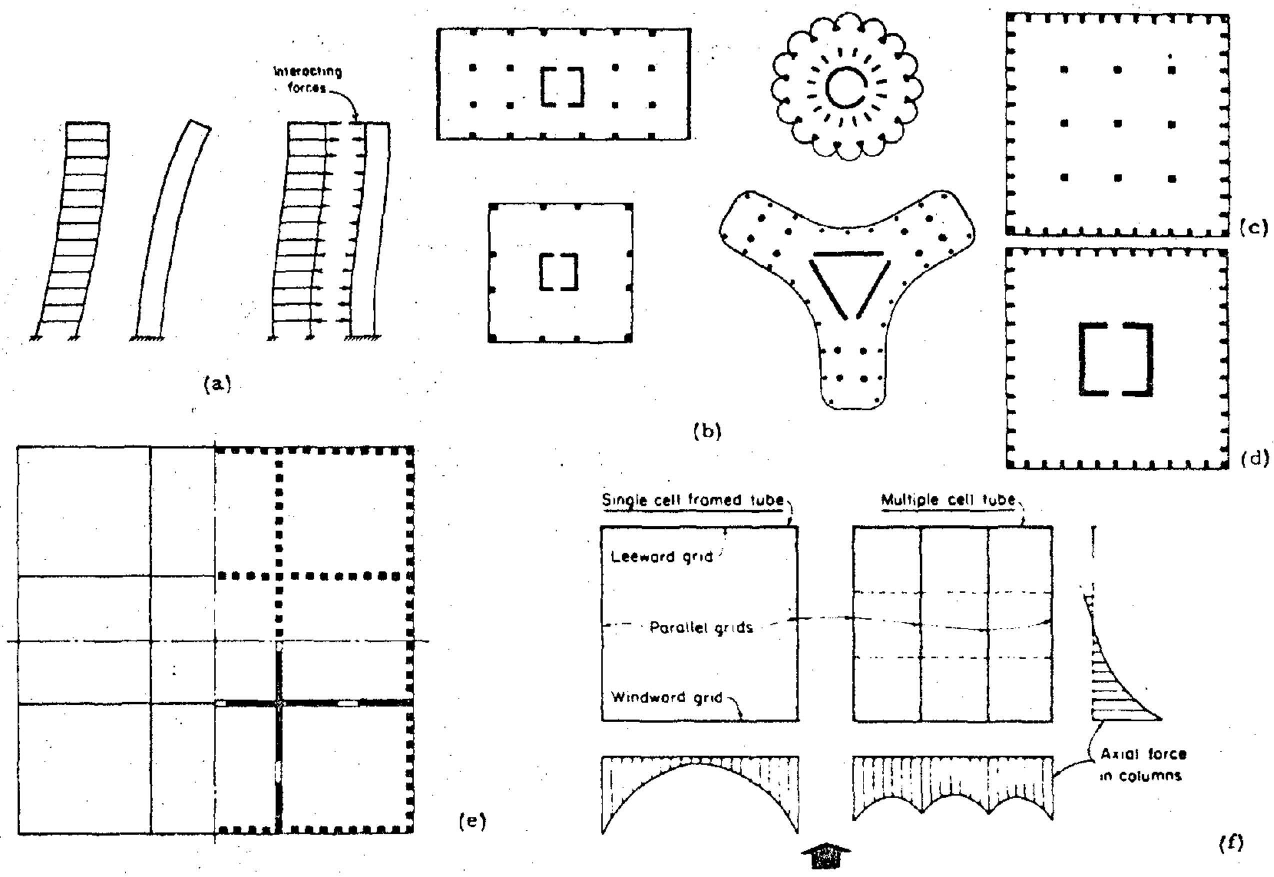


Fig. 1: Lateral Load Resisting Structural Systems

For still taller structures, especially where a large plan area is involved, intersecting planes of interior walls or closely spaced column-beam grids traversing the entire width of the building may be used (Fig. 1e). Connecting the interior walls with the exterior peripheral grids reduces the shear lag* across the windward and leeward grids and allows the latter to participate to a greater extent in resisting the lateral load (Fig. 1f). The use of such interior vertical diaphragms when indicated, essentially produces a vertical multi-cell cantilever box beam [1,3].

Table 1: Guide to Selection of Structural Systems

Structural Systems	Number of stories*	
	Office Buildings	Apartment Buildings hotels, etc.,
Frame	up to 15	up to 20
Shear Wall (egg crate)		up to 150
Staggered Wall Beam		up to 40
Shear Walls acting with Frames	up to 40	up to 70
Single Framed-Tube	up to 40	up to 60
Tube-in-tube	up to 80	up to 100

*The values given here are based on present day practice as well as trends indicated by current thinking.

Table 1, reproduced from [4], is presented as a guide in the choice of an appropriate structural system for a new building. The ranges of suitability shown may vary somewhat depending upon the use of the building, the story heights, and the design live and wind loads.

EARTHQUAKE RESISTANT DESIGN

Structural Response to Earthquakes

The effects of earthquakes may be due directly to the causative process, such as faulting or volcanic action; or due to the ground motion resulting from the passage of seismic waves. Of the latter effects, two types can be distinguished: one in which dynamic (inertial) effects are predominant; and the other, associated with landslides, soil consolidation or liquefaction triggered by earthquake motions, where differential inertial effects within a structure are negligible. Except in unusual circumstances, most of the damage associated with earthquakes has been the result of dynamic effects, and engineering efforts aimed at designing earthquake-resistant structures are concerned mainly with such effects [5].

The ground motion at a particular site is influenced by three factors: (a) source parameters, such as the earthquake magnitude (energy released), depth of focus and geological conditions at and near the focus; (b) transmission path parameters, i.e., epicentral distance and properties and geological character of the intervening ground; and (c) local site parameters, or the geological configuration and properties of the ground at the site.

The forces induced in a structure by an earthquake result directly from the distortions produced by the ground motion. A simplified picture of the behavior of a building during an earthquake can be obtained by considering Fig. 2 [5]. As the ground on which the building rests is displaced, the base of the building moves with it. However, the inertia of the building mass resists this motion and causes the building to suffer a

*The decrease in the vertical forces transmitted to the columns as one moves from the corner toward the center of a frame subjected to lateral loads.

distortion (greatly exaggerated in the figure). This distortion wave travels along the height of the structure in much the same manner as a stress wave in a bar with a free end. The continued shaking of the base causes the building to undergo a complex series of oscillations.

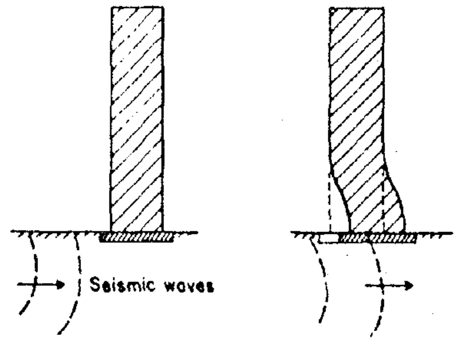


Fig. 2: Effect of Ground Motion on Structure

Design Criteria

The performance criteria implicit in most earthquake code provisions [6] require that a structure to able to:

a. Resist earthquakes of minor intensity without damage (within the elastic range of stresses),

b. Resist moderate earthquakes with minor structural and some nonstructural damage, and

c. Resist major catastrophic earthquakes without collapse.

While no clear quantitative definition of the above earthquake intensity ranges has been given, their use implies the consideration not only of the actual intensity levels, but also of their associated probabilities of occurrence with reference to the expected life of a structure. The quantitative definition of such earthquake intensity ranges would have to consider all the significant ground motion characteristics affecting structural response, i.e., the magnitude of acceleration pulses, frequency characteristics, and duration of the significant portion of the ground motion. The recurrence interval associated with each intensity range would then have to be established for each particular site. The present lack of adequate data on earthquakes renders such an approach beyond immediate realization. The principal concern in earthquake-resistant design is the provision of adequate strength and ductility for the most intense earthquake which may reasonably be expected at a site during the life of a structure, as well as the provision of adequate stiffness for damage control under more moderate earthquakes.

Possible Control of Seismic Input

Certain features (e.g. symmetry, absence of major discontinuities, etc.) in a structure are desirable in that they reduce sharp peak concentrations of earthquake-induced forces. These aspects are discussed in a later section. The generally desirable objective of reducing the seismically induced forces in a structure can logically be pursued further by introducing special devices or mechanisms into the structure. This approach has so far been limited to a very few applications. With the hope that further research will develop the full potential of this possible course, it is briefly reviewed in the following paragraphs.

The schemes which have been proposed in the past to reduce the effect of ground motion on the structure fit mainly into two categories: isolator devices and absorber-damper systems.

Isolator devices--These mechanisms are intended to separate the structure completely from its foundation, using rollers, friction pads, or water. Isolators act essentially as force-limiting devices, since the base shear force in the structure cannot exceed the limiting friction force in the isolating mechanism. To be practical, an isolator mechanism should satisfy the following conditions [7,8]:

1. Relative displacements across the mechanism should be allowed in all directions, but should be limited to certain tolerable values.

2. Post-earthquake residual displacements should be minimized.

3. Wind should not cause relative motion across the mechanism.

4. Preferably, some energy should be absorbed by the mechanism.

5. Impact type forces generated at the end of the operative range of the mechanism should be minimized.

6. Manufacture, maintenance and installation of the mechanism should be as inexpensive as possible.

Requirement 2 above poses the most problems with practical isolation devices. Many apparently feasible solutions to this problem have been proposed [7], and isolators have been used extensively in the case of machine foundations [9]. To date, at least one five-story reinforced concrete building in Mexico City has been isolated from its foundation by a ball-bearing system [10]. A limited amount of research work on isolator systems is currently in progress in the U.S. [11].

Absorber-damper systems--In general, it appears feasible to control the earthquake ground motions input to the base of low, rigid buildings by means of an isolator mechanism, or by taking advantage of the properties of the surrounding soil and its interaction with the foundation [9].

In the case of tall, slender buildings, the control (isolator) system must perform two functions [9,12]. First, it must prevent the build-up of unacceptably large accelerations which may occur as a consequence of resonance in one of the higher modes of the building when it is excited by the high frequency components of the ground motions. Second, it must prevent the development of large deformations in the building which may occur as a consequence of its fundamental mode having been excited by the low frequency components of the ground motion. One way to realize these goals is to increase the damping of the structure to avoid sharp peak values of response. Another way is to confine the energy absorbing function of the structure to built-in special devices or to specially designed portions of the structure, which would absorb and dissipate large volumes of energy through multilinear elastic or elasto-plastic behavior.

The use of elasto-plastic nonlinearity has often been proposed, although such use may not be desirable at all times, since the period of the structure tends to become longer just when the excitation of low frequencies is predominant. Studies have indicated that devices based on the plastic torsion of mild steel bars can provide large energy absorption with adequate fatigue resistance. A practical device based on this principle has been developed and is currently undergoing full-scale tests in New Zealand where it will be incorporated into the piers of a reinforced concrete railway bridge [13]. Another technique for controlling the earthquake energy input to a structure was originally suggested in 1929 [14] and involves the use of a flexible and soft (relatively low yield strength) first story (or lower stories) [15]. During an earthquake, predetermined areas in the lower levels of these structural systems are supposed to undergo bilinear forms of elasto-plastic hysteresis, thereby absorbing most of the earthquake energy. Portions of a building above the soft story then need only be designed for gravity and wind loads, as in nonseismic zones. An 11-story reinforced concrete hospital based on the above principle, designed by ABAM Engineers, has been built in Tacoma, Washington. The presence of a restoring force within the system prevents instability due to large distortions of the soft story. In general, the possible residual displacements associated with elasto-plastic hysteresis may be a drawback of absorber systems based on such hysteresis.

The "double-column" or "multi-column" system, proposed by Japanese researchers, represents one of the ways to produce elastic nonlinearity in a building [7,12]. When multi-columns are used, only the inner columns support the axial and bending stresses until the deformations are large, when the outer columns also share the stresses, thus producing hardening-spring type stiffness. The nonlinear characteristics can be adjusted through the gaps between the inner and outer columns. This system, in conjunction with suitable dampers, is planned for installation in the lower three stories of the 200m tall Yosuda-Kasai Building in Tokyo [12].

The modern trend in research on input control systems is towards 'active control', the aim being to develop control mechanisms which are regulated by electronic signals from sensors of displacements, velocities, accelerations or forces [8,16].

One of the main problems associated with the use of isolator and/or absorber-damper mechanisms concerns their reliability. Use of earthquake simulators (shaking tables) appears to be essential in reliability studies. Tests of full-scale models are not possible with the presently available earthquake simulator facilities, except for every small structures. It is also doubtful whether such tests could be carried out even with the largest conceivable shaking table that could be built in the near future [9].

With or without the use of input control devices, the design of earthquake resistant structures must meet the twin requirements of safety or prevention of collapse and damage control. These requirements will now be discussed separately, and in some detail.

Need for Ductility

In most reinforced concrete structures, it is uneconomical to resist the forces, generated during strong seismic ground excitations, within the limits of elastic response of the structure. It is accepted that during rare ground accelerations of large intensity, yielding and consequent plastic deformations may occur at some or all critical areas within the structure. Because prevention of collapse is a fundamental design criterion, it is necessary to ensure that the post-elastic deformations in all parts of the structure can occur while the lateral and vertical load capacities of the structure are substantially maintained [17].

The ability of a structure to deform past the elastic limit is usually measured in terms of ductility. Ductility in reinforced concrete structures in general is defined as the ratio of a specified distortion at a particular stage of loading to that at the onset of yielding. With certain restrictions, the term "ductility" may be considered as a useful index of the suitability of a structure for seismic resistance.

Fig. 3 shows the lateral force versus lateral displacement relationship for two structures with identical stiffness, but of differing strengths, responding to the same earthquake. Structure A is able to respond to the given earthquake completely within the elastic range. The maximum deflection corresponding to the full elastic response is Δ_A. In structure B, when the lateral force reaches F_B, the structure reaches its elastic limit at a lateral displacement of Δ_B.

It has been shown 18-20 that whether the response is elastic or inelastic, the displacements which regular medium-to-highrise structures undergo, when subjected to a typical earthquake motion, are of the same order of magnitude. Thus, structure B must be able to deform plastically from B to B', if it is to survive the earthquake. It can be seen that the designer may select a lower strength than the elastic reponse force (F_A), provided that inelastic deformations and the resulting damage are acceptable. The current building codes specify a design load (F_B) 1/3 to 1/6 of the force required to resist the earthquake elastically (F_A).

This reduction of the strength requirement is justified only if accompanied with design and detailing requirements for the structure to be ductile, so that it can deform plastically without collapse. In the above comparison, Δ_A is the same as Δ_u, the ultimate deflection of

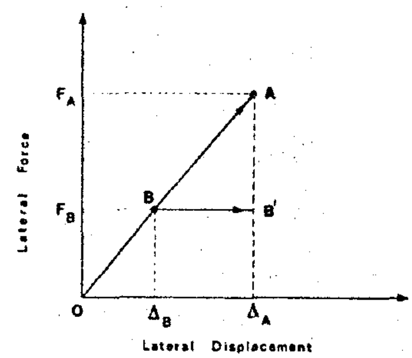


Fig. 3: Force-displacement Relationships of Elastic and Inelastic Structures

structure B, and Δ_B is the same as Δ_y, deflection at yielding of structure B. The ratio $\Delta_A/\Delta_B = \Delta_u/\Delta_y = \mu_\Delta$ is the displacement or system ductility factor for the structure.

Distribution of Ductility Requirements Along Structure

It is important to draw a distinction between the ductility factors associated with the lateral displacement of a structure and local ductility factors. Since the former is achieved through inelastic deformations at the critically stressed portions of a relatively few members, the corresponding local ductility factors are of primary interest in design. Thus, it is worthwhile presenting here some of the more significant results of analytical studies of the earthquake response of frame, wall and frame-wall structures.

Frames--The configuration and relative member stiffnesses of the basic 20-story frame structure considered in a study by Clough and Benuska [21], from which most of the results presented here are drawn, are shown in Fig. 4a. The frames were designed for vertical loads plus the lateral forces prescribed by the Uniform Building Code, 1964 Edition. The yield moments were taken as twice the corresponding computed design values for the girders and six times the corresponding design values for the columns.

In nonlinear dynamic response analyses, the moment-rotation characteristics of the members were assumed to be of the bilinear type, with the post-yield branch having a slope equal to 5% that of the elastic branch. The term ductility factor was defined as the ratio of the maximum rotation at the end of a yielded member, to the yield rotation angle. The yield rotation angle was defined as that corresponding to a moment acting at the end of a simply-supported member having the same section but a span equal to half that of the actual member. The use of a half-span was based on the antisymmetrical mode of deformation of frame members due to lateral displacement.

The results shown were obtained by subjecting the base of the structures to the first 4 sec. of the 1940 El Centro earthquake (N-S component). Other earthquake records with different frequency characteristics may produce results significantly different from those presented.

1. Comparison of linear (elastic) and nonlinear response--Fig. 4b(i) shows that the maximum lateral displacements for both the elastic and the nonlinear frames are approximately equal, as schematically indicated earlier in Fig. 3. This similarity, however, does not imply the develop- ment of similar maximum deformations in corresponding members of the two frames. Fig. 4b(ii) shows the girder ductility requirement for the non- linear case varying from 2 at mid-height to 5 at the top, compared to a maximum-to-yield moment ratio of about 2 for the elastic case. An analysis assuming completely elastic response slightly overestimates the inelastic deformations in the columns (Fig. 4b(iii)). In Fig. 4, as well as in subsequent figures, a ductility factor less than unity indicates the ratio of the maximum moment to the yield moment in a member.

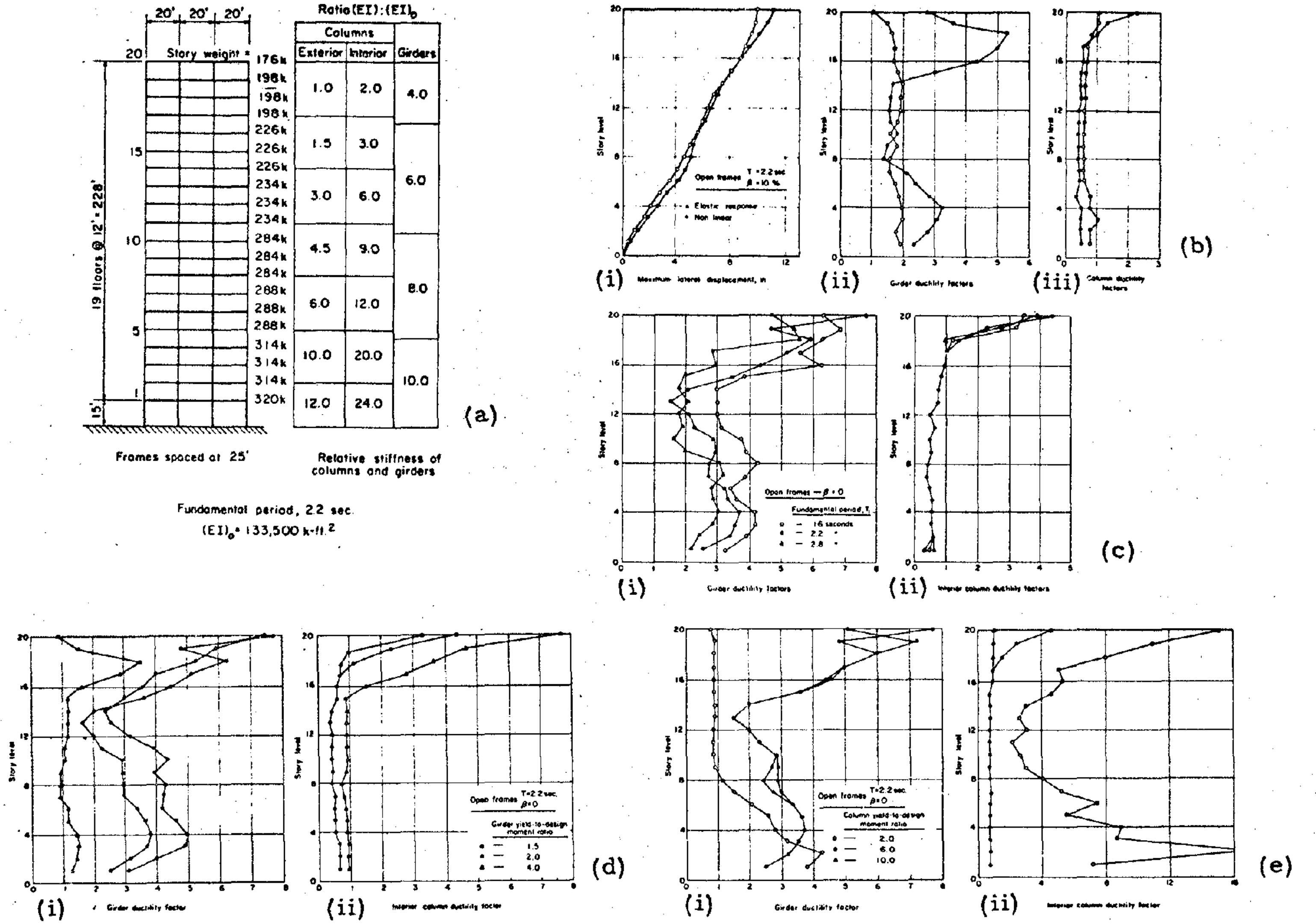


Fig. 4: Seismic Response of Frame Structures

2. Effect of period of vibration--Two 20-story frames having fundamental periods of 1.6 and 2.8 sec. were considered in addition to the standard 2.2 sec. frame. The basic stiffness parameter, $(EI)_o$, was varied to obtain the different periods. The results, shown in Fig. 4c, indicate that there is a slight decrease in girder ductility requirements for the more flexible (long-period) structures. However, a study by Goel and Berg [22] of the response of 10-story, single bay frames to three different earthquake records showed that this particular trend can be reversed in the case of earthquake records characterized by dominant velocity spectrum peaks in the 2-3 sec. range.

A probabilistic study by Ruiz and Penzien [23] of the response of 8-story, shear-beam models subjected to a number of artificially generated accelerograms showed that in stiff, short-period structures with a fundamental period of about 0.5 sec., the ductility requirements tend to decrease toward the top of the structure. This contrasts with the variation typical of more flexible frames shown in Fig. 4c where the influence of the higher modes of vibration causes a significant increase in the ductility requirements in the top stories. Ruiz and Penzien also observed that the ductility requirements at the base of a stiff structure are significantly greater than those for a flexible structure subjected to the same excitation.

3. Effect of strength of girders--Three frames were considered: the reference structure with a girder yield-to-design moment ratio of 2.0 and two other frames, identical to the first, except that the yield moments were 1.5 and 4.0 times the design moments.

As expected, the girder ductility requirements decreased with increasing girder strengths. This is shown in Fig. 4d(i). More significant, however, is the fact that the increase in girder strength forced more of the inelastic deformation to occur in the columns, as indicated in Fig. 4d(ii). In general, decreasing the yield strength of one member type, i.e., columns or girders, with respect to another tends to attract inelastic deformation toward the weaker members, resulting in reduced yielding in the stronger member type.

4. Effect of column strength--This variable was studied by considering two frames, having column yield-to-design moment ratios of 2.0 and 10.0, in addition to the reference frame which had a moment ratio of 6.0. Fig. 4e indicates that increasing the column strength beyond that corresponding to a ratio of 6.0 does not materially affect the response. This follows from the fact that the columns in the reference building remain essentially elastic during the response.

Fig. 4e, however, shows that a reduction in column strength can have a significant effect on the distribution of ductility requirements. If the columns do not have a sufficient margin of strength above the design level, most of the inelastic deformations will tend to occur in the columns. Because of the danger of instability associated with excessive yielding in the columns, such a condition should be avoided.

Walls--The results presented are from the report of a recent investigation conducted at the Portland Cement Association (PCA) [24]. The basic structure considered in this study is a 20-story building consisting mainly of a series of parallel walls (Figs. 5a-c).

The stiffness of the wall in the basic building was assumed uniform along the height. A constant wall cross-section throughout the height was also assumed. However, a reduction in the yield strength of sections above the base was included to reflect the effects of axial loads on moment capacity. The building was assumed to be fully fixed at the base. Inelasticity was allowed in dynamic analyses by means of concentrated flexural 'point hinges' which formed at the ends of elements when the yield moment was exceeded at these points. The hysteretic moment-rotation relationship for these hinges was an extended version of Takeda's model [25] which accounts for the observed decrease in reloading stiffness in reinforced concrete members subjected to reversed inelastic loading. A 12-mass model of the 20-story walls was used in analyses (Fig. 5d), with the masses concentrated at each floor level in the first four floors where most inelastic action usually took place.

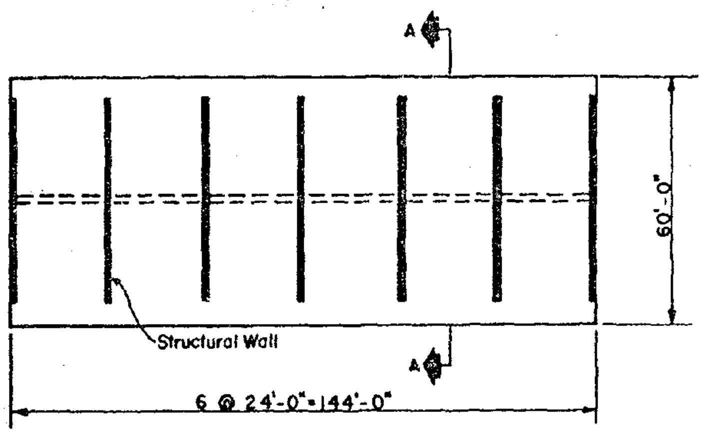
The ground motion used in analyses had the same frequency characteristics as the E-W component of the 1940 El Centro record. The duration of the motion was set at 10 sec. The intensity was normalized to 1.5 times the spectrum intensity corresponding to the first 10 sec. of the N-S component of the 1940 El Centro record.

Ductility was defined on the basis of nodal rotations as being equal to θ_{max}/θ_y where θ_{max} was the maximum computed rotation at the node, and θ_y was the nodal rotation corresponding to yielding at the base.

1. Effect of fundamental period--The effect of the initial fundamental period was investigated using values of 0.80, 1.40, 2.00 and 2.40 sec. to cover the practical range for 20-story buildings. Each period was investigated under varying values of yield strength of the critical section at the base (M_y), in order to examine the relationship between these two major variables and the response quantities. Fig. 5e presents ductility requirements along the height of the walls for M_y = 500,000 in-k. The ductility requirements become greater with decreasing fundamental period (increasing stiffness). Beyond a certain value of the fundamental period, however, the ductility requirements do not decrease significantly with an increase in period.

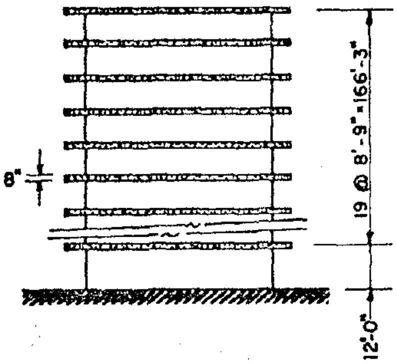
2. Effect of flexural strength--The values considered for the yield strength of the base critical section ranged from 500,000 to 1,500,000 in-k. The results, for the particular case of T_1=1.4 sec. are presented in Fig. 5f. It can be seen that the ductility requirements increase significantly as the yield level decreases.

Frame-Wall Systems--The results presented here are from the Clough-Benuska study referred to earlier [21]. Fig. 6a shows the relative stiffnesses of the members of the standard structure in terms of a reference $(EI)_0$. The value of $(EI)_0$ has been adjusted to give the standard



PLAN

(a)

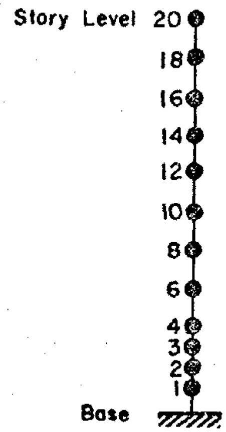


SECTION A-A

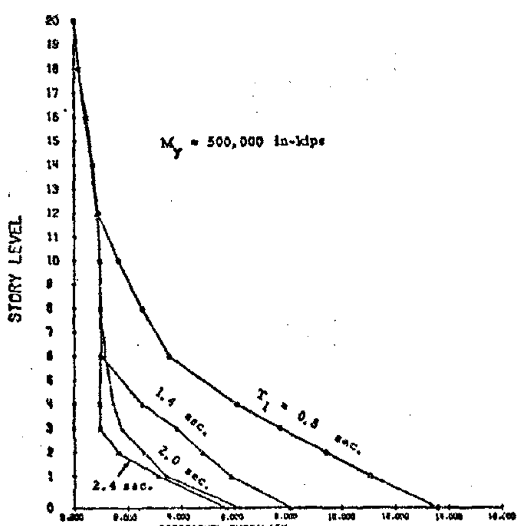
(b)

Number of stories		20
Height		178.25 ft.
Weight (for mass computation)		4370 k/wall
Stiffness EI for T_1 =	0.80 sec	6.34×10^{11} k-in^2/wall
	1.40	2.052×10^{11}
	2.00	0.990×10^{11}
	2.40	0.684×10^{11}

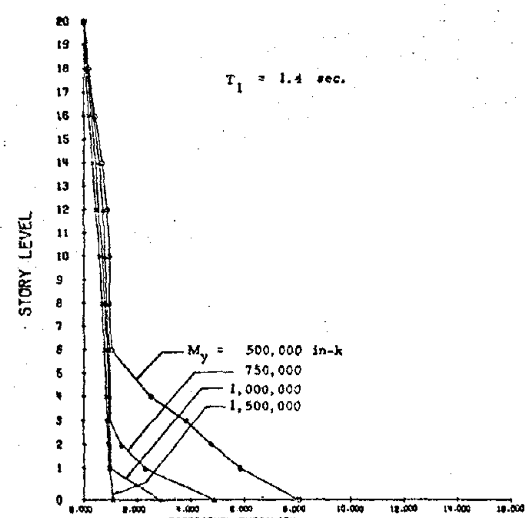
(c)



(d)



(e)



(f)

Fig. 5: Seismic Response of Isolated Structural Walls

structure a fundamental period of 2.2 sec. As with the frame building discussed earlier, the design moments were determined by a computer analysis for the static vertical and the code seismic forces.

1. Effect of design assumptions concerning distribution of lateral loads between frame and structural wall--Designs corresponding to three different ways of distributing lateral loads, found in practice, were considered. A first design was based on the assumption that the entire lateral load was carried by the structural wall. The frame for this building was designed only for vertical loads, with the girder moments being uniform and the column moments increasing from top to bottom. A second design was based on the Uniform Building Code provisions requiring the frame to be designed for (at least) 25% of the total lateral forces. This led to girder and column moments, both of which increasd from top to bottom. A third design was based on the true interactive behavior of the frame-wall system. In this case, the girder and column moments were largest at mid-height and decreased both upward and downward. In each case, the ratio of yield-to-design moments was set equal to 2 for the girders and 6 for the columns and walls. In all cases, the reference stiffnesses were adjusted to yield a fundamental period of 2.2 sec.

The girder and column ductility requirements corresponding to the three buildings considered are shown in Fig. 6b. Also shown is a curve corresponding to the 25% lateral-load frame building with the structural wall hinged at the base. The very favorable distribution of strength in the interaction frame building, resulting in significantly lower ductility requirements for girders over the entire height and for columns at the top, is evident. The relatively low design strength of the frame in the gravity-load frame building is reflected in the high girder ductility requirements. It is worth noting that designing for frame-wall interaction tends to eliminate yielding of the columns at the top stories.

Fig. 6b shows that the ductility requirements (M_{max}/M_y for ductility ratios less than unity) in the structural walls for the four buildings considered are roughly of the same order of magnitude. For the yield-to-design moment ratio assumed, none of the structural walls was stressed beyond the elastic range.

2. Effect of period of vibration and frame-to-wall stiffness ratio--Two structures with fundamental periods of 1.6 and 2.6 sec. were considered in addition to the reference 25% lateral-load frame building, having a fundamental period of 2.2 sec. The 2.6 sec. building had a 10-ft wide structural wall with a stiffness ratio relative to the wall in the reference building of 0.2, while the 1.6 sec. building had a 38-ft wide wall and a stiffness ratio of 5.0.

Fig. 6c shows the girder and column ductility requirements. There is no significant difference in column ductility requirements among the three buildings. A slight decrease in girder ductility requirements occurs in the stiffer (shorter-period) structures. This trend is contrary to that observed in open frame structures. It should be realized that the periods of the three structures considered differ, not because of a change in stiffness of the frames (as was the case with the frame structures

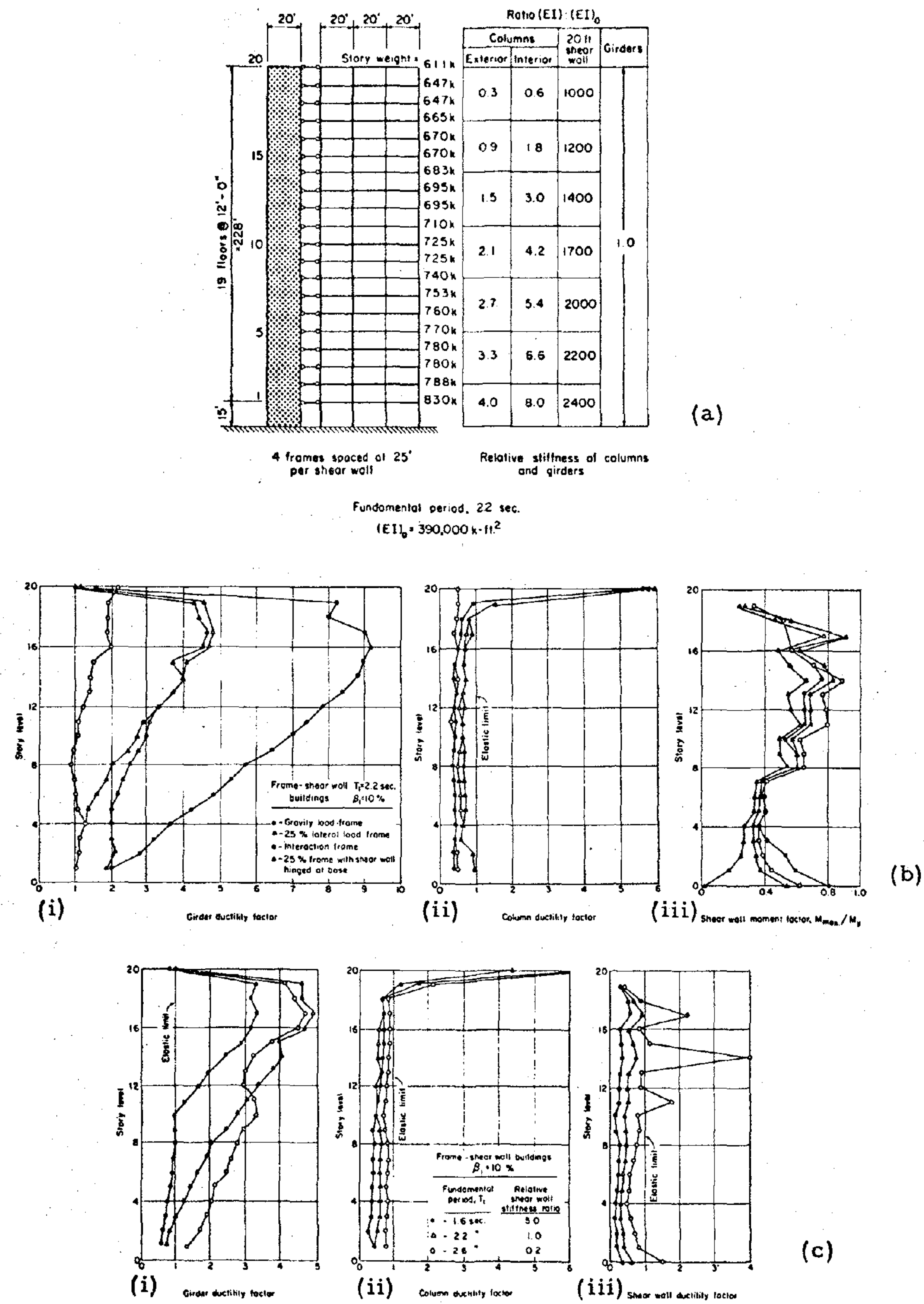


Fig. 6: Seismic Responses of Frame-Wall Structures

discussed earlier), but because of a change in the width and hence the stiffness of the structural walls. In all three structures, the frame portions were identical. The observed difference in girder ductility requirements can thus be interpreted as reflecting the effect of the wall-to-frame stiffness ratio rather than of the period of vibration.

A plot of the structural wall ductilities, shown in Fig. 6c, indicates a decreasing ductility requirement for the stiffer structures. More important, however, is the relatively large ductility requirement indicated for the 2.6 sec. structure, compared to the elastic behavior of the other two structures. This points to the potential danger of rupture in such stiffening elements in structures with low wall-to-frame stiffness ratios.

Design for Prevention of Collapse

The design of a structure for prevention of collapse usually consists in proportioning and detailing the critical regions such that they possess adequate strength and ductility. The discussion in this section is thus focused on the critical regions, rather than on the structure as a whole.

One important design consideration affecting the integrity of the entire structural system is the provision of 'multiple lines of defense'. This can be accomplished with a high degree of static indeterminacy and with the establishment of an advantageous sequence in the propagation of yielding. This aspect is discussed in a later section.

While there have been attempts to relate code-specified minimum system ductilities and local ductility requirements in the case of frames [26] and cantilever walls [17,27], the best way to assess the local ductility requirements in the critical regions of a particular class of structures, corresponding to a specified earthquake intensity, is to carry out dynamic inelastic analyses of structures representative of the class under various combinations of structural and ground motion parameters. This was done at PCA in the case of isolated wall structures [24,28], resulting in charts such as the one illustrated in Fig. 7. The chart gives the required ductilities, based on nodal rotations at the first floor level, in 20-story structural walls under an earthquake with a spectrum intensity equal to 1.5 times that of the first 10 sec. of the N-S component of the 1940 El Centro record. The ductility requirements decrease with increasing flexural yield strengths of the base critical section, as well as with increasing periods. There will, in practice, be an upper limit on the period or flexibility (as discussed later). One is thus faced with a situation of trade-off between flexural strength and ductility requirements.

It must be understood that the critical regions have to be designed such that the required ductilities are attainable in the presence of the shear and the axial stresses which the regions are called upon to carry. The presence of shear has a decidedly adverse effect on ductility. While axial loading is known to have a detrimental effect on the curvature ductility of a section [29], its effect on the rotational ductility of a member segment is not necessarily harmful. This is because the presence of axial loads results in an enlarged concrete compression zone capable of transmitting shear. This has a delaying, if not preventive, effect on the

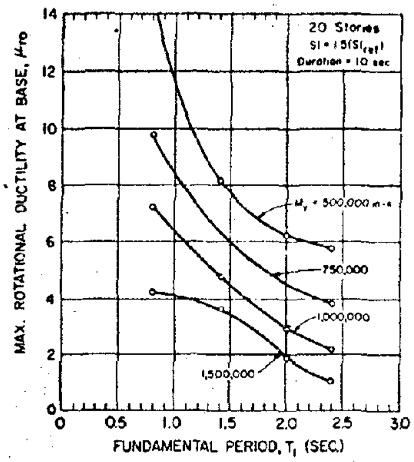


Fig. 7: Flexural Strength and Ductility Demands in Isolated Walls

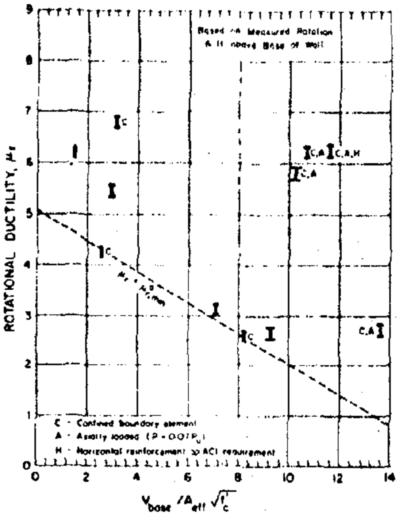
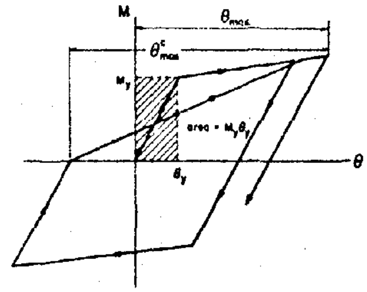


Fig. 8: Rotational Ductilities Available in Hinging Regions of Isolated Walls

possible occurrence of nonductile shear failure. Rotational ductility is thus enhanced by axial loading, particularly in the presence of high shear. Axial loading, to have any beneficial effects, however, has to be limited in magnitude to rather moderate levels. High axial loads, tending to cause compression failures, are bound to have harmful effects. Fig. 8, based on recent tests carried out at PCA [30], may serve as a guide to rotational ductilities available in reinforced concrete wall segments. The beneficial effect of confinement reinforcement on ductility should be noted. More experimental research is needed in this area to establish minimum available ductilities as functions of axial and shear stresses, under various combinations of sectional parameters (shape, amounts of flexural, confinement and shear reinforcement, etc.). Such research must also extend to critical regions of frames and frame-wall systems.

In [28], the possibility was raised that designs based on a comparison of just one measure (e.g., rotational ductility) of deformability demand and deformation capacity may not be entirely safe, particularly since the estimates of deformation capacity are usually based on laboratory loading histories which are different from those experienced by critical member segments under seismic conditions. Thus, in addition to rotational ductility as defined in Fig. 9, three other measures of deformation as well as energy dissipation (also defined in Fig. 9) were considered. Based on a comparison between estimated available and required values of the various quantities under severe earthquake conditions, it was determined [28] that designs satisfying minimum deformability (energy dissipation) requirements in terms of rotational ductiity will also be safe with regard to the other measures of deformation and energy dissipation.

In addition to ductility, sufficient shear strength for the critical segments must also be provided for in design. This may not always be simple, since repeated reversed loading of reinforced concrete member segments in the inelastic range may lead to a reduction in their shear resistance. There is a paucity of test results on which one can base reasonable estimates of the shear capacity of reinforced concrete member segments subjected to repeated reversing loads of large amplitude [31]. Partly because of this, and partly out of concern for ductility as well as energy dissipation capacity (which suffers because of stiffness degradation caused by moment reversals in the presence of moderate-to-high shear), Paulay [32] has made the following suggestion for the case of tall (height/depth > 3) structural walls: "Where it is essential that the lateral and gravity strength be maintained in a ductile manner, ...every attempt must be made to suppress a shear failure. This is only possible if the shear force, associated with the maximum possible flexure strength of the critical section, taking into account the increased yield strength of the flexural reinforcement due to strain hardening, is provided for in such a way that the shear (web) reinforcement will not yield." The following suggestions concerning low (height/depth < 2) structural walls have also been made:



Measures of Deformation:

rotational ductility, $\mu_r = \dfrac{\theta_{max}}{\theta_y}$

cyclic rotational ductility, $\mu_{rc} = \dfrac{\theta^c_{max}}{\theta_y}$

cumulative rotational ductility, $\Sigma\mu_{rc} = \dfrac{\Sigma\theta^c_{max}}{\theta_y}$

cumulative rotational energy, $\Sigma A_r = \dfrac{\text{cumulative area under } M-\theta \text{ loops}}{M_y\theta_y}$

Fig. 9: Definitions of Ductility and Energy Dissipation Capacity

(a) If a ductile (i.e., flexural) failure mechanism is desired, then the nominal shear stresses, associated with the maximum possible flexure strength of the critical section, must be moderate say, $v_u < 5\sqrt{f'_c}$ psi.

(b) Because the flexural failure mechanism is associated with large cracks, no reliance can be placed on the concrete within the hinging region in contributing towards shear strength. Consequently, in the hinging region the whole of the shear force should be resisted by stirrups.

Bertero and Popov [33,34] recommend that flexural members in general be designed such that their maximum bending strength does not require the development of maximum average nominal shear stresses beyond $3.5\sqrt{f'_c}$ psi. If it is not possible to keep the nominal shear stress below this level, special web reinforcement beyond that required by present code provisions should be used. Even then, the maximum nominal shear stress should not exceed $6\sqrt{f'_c}$ psi if two or more load reversals at a displacement ductility ratio of 4 or greater (for the member) is expected. The maximum nominal shear stress, in any case, should preferably be confined to a value considerably lower than $10\sqrt{f'_c}$ psi.

DAMAGE CONTROL

The provision of adequate local ductilities and shear strength in all critical regions will not only minimize the probability of collapse, but will usually also minimize earthquake damage to the structural elements of a building. Careful attention must be paid to the detailing of joints and to proper anchorage of all reinforcement. In addition, considerations such as (1) the avoidance of unnecessary torsion and force concentrations, (2) proper tying together of structural elements, (3) prevention of hammering, and (4) taking proper account of stiff infills in spaces between frame elements or columns, are also important. These considerations are discussed in the next section. The discussion here is on damage to nonstructural elements, which is of utmost concern, since such elements represent a major portion of the cost of residential and office buildings.

The magnitude of interstory horizontal deformations appears to be the prime factor determining the amount of earthquake damage to nonstructural elements.

Force-Deformation Characteristics of Nonstructural Elements

Nonstructural elements can be made of brittle or ductile materials, each characterized by its own response to loading.

Brittle elements such as unreinforced masonry partitions, glass panes, etc. fail abruptly after reaching their maximum strength. Depending upon the magnitude of their deformation before sudden brittle failure, they can be either relatively rigid (asbestos cement sheets) or relatively flexible (gypsum drywall panels).

Ductile elements reach their maximum strength and continue to deform while maintaining an acceptable load level. Many of the brittle materials can be made into ductile elements, either by reinforcing them (i.e., reinforced masonry), or by proper assembly of units (i.e., walls made of individual gypsum drywall panels with flexible connections between them).

While for some nonstructural materials and assemblies the force deformation characteristics are known, no information exists for many other elements. This lack of information makes it difficult, if not impossible, to establish rational limitations on interstory distortions. Research is needed to establish force-deformation characteristics for all nonstructural elements incorporated into earthquake-resistant buildings.

Design and Detailing for Damage Control

If the nonstructural elements are ductile and are thus able to distort and accommodate the elastic and plastic distortions of the structure without cracking or breakage, then no special detailing is required; these elements will not suffer any significant earthquake damage.

Where brittle nonstructural elements are used, they can be protected against earthquake damage by using them in conjunction with rigid structures having interstory deformations restricted to a level which can be tolerated by the brittle elements. This can be accomplished in buildings incorporating structural walls, except for the hinging region in which shearing type deformations may be large. Within the anticipated hinging region, special detailing for brittle nonstructural elements may be required.

In flexible structures (frames) with expected interstory deformations larger than the damage deformation capability of brittle nonstructural elements, such brittle elements should be detailed so as not to be strained when the frame distorts in an earthquake. Partitions can be made "floating", window panes can be embedded in neoprene gaskets, and mechanical appurtenances can be specially detailed. The amount of expected deformation can be determined from analysis by considering the combined elastic and inelastic story deformations.

Although generally the only adverse effect of the required special details is to add to the cost, there are instances where performance is affected, as when accoustic problems are caused by floating partitions.

PLANNING AND DESIGN CONSIDERATIONS

A reasonably good basis for a preliminary design of an earthquake-resistant building would be a structure proportioned to satisfy the requirements of gravity and wind loading. The planning and layout of the structure, however, must be undertaken with proper consideration of the dynamic character of earthquake response. Thus, modifications in both configuration and proportions to anticipate earthquake requirements may be incorporated immediately into the design for gravity and wind. The following are some of the design considerations.

1. Drift limitation--A limitation on drift or lateral deflection due to wind is the principal criterion used in assessing the proper lateral stiffness to be built into tall buildings, and may determine the type of structural system to be employed. The use of a maximum allowable drift is based on the need to limit to safe or tolerable levels the effects of lateral sway on (a) the stability of individual columns as well as the structure as a whole, (b) the integrity of nonstructural elements, and (c) the comfort of the occupants. The precise relationship between drift due to wind and the above three factors remains to be established. To date only the Uniform Building Code [35], and the National Building Code of Canada [36], among the North American model building codes, specify a maximum drift of H/500 (H = building height), corresponding to the design wind loading. Also, ACI Committee 435 recommends a drift limit of H/500 [37]. The present day design of tall reinforced concrete buildings containing structural walls provides extremely rigid structures [38] with a drift (computed by advanced methods) between H/1000 and H/2500, depending on the slenderness ratio of the building and the number and layout of walls.

Basically the same considerations as mentioned for wind enter in aseismic design, although one might expect slightly more liberal drift limitations under major earthquakes. For severe earthquake motion, the principal consideration insofar as drift is concerned is the stability of the structure under the action of gravity loads when undergoing large lateral displacements. The SEAOC Code [6] mentions an allowable drift due to the specified earthquake forces twice that allowed for wind. In applying such a limit, a distinction should be made between the drift produced by the code-specified static forces and the dynamic lateral displacements corresponding to a particular earthquake. The latter could be several times larger than the former [21]. It also follows from the previous section that the need for damage control may require a limit on the interstory drift as well, although no specific limits have been suggested.

Fig. 10 based on the PCA study on isolated walls [28] shows the maximum drift and interstory drift in 20-story walls subjected to intense earthquakes (spectrum intensity = 1.5 times that of the first 10 sec. of the N-S component of the 1940 El Centro record), as functions of the fundamental period and the rotational ductility available in the critical region at the base (extending to the first floor level). The displacements shown are envelope values of the displacements caused by a number of earthquakes of varying frequency characteristics. It can be seen that if and when suitable limits on the drift as well as the interstory drift can be decided on, a corresponding allowable upper limit on the fundamental period can be established. The latter can, in turn, be translated into an allowable lower limit on the flexural stiffness.

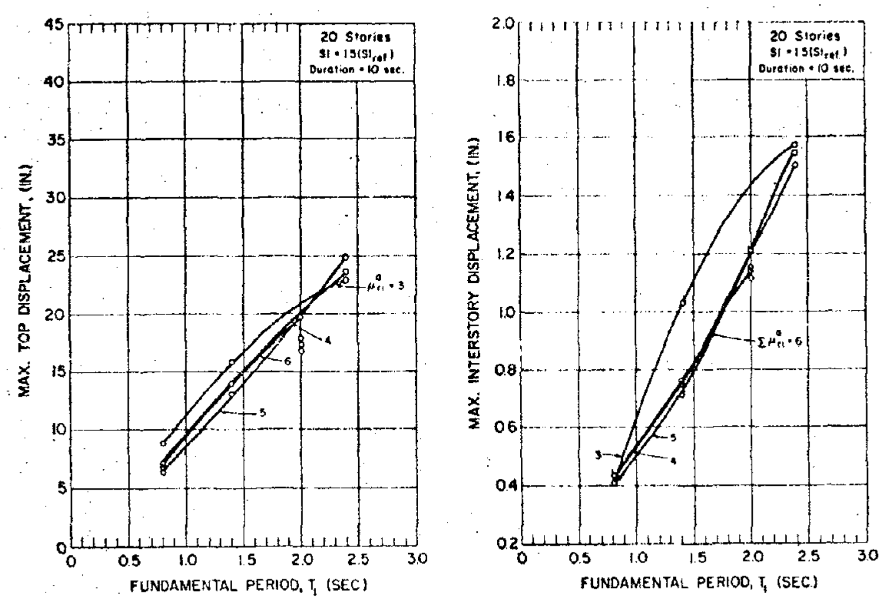


Fig. 10: Maximum Lateral Displacements and Interstory Displacements in Isolated Structural Walls

2. Avoidance of unnecessary torsion and force concentrations--A building which is simple in both plan and elevation, with a minimum of setbacks or changes in section, is generally preferable to an irregularly shaped structure. This is because the effects of force concentrations which occur at major discontinuities in either geometry or stiffness even under static loading, tend to be aggravated under dynamic conditions. The required ductility at such regions of discontinuity is usually substantially greater than at other portions of a structure.

Although it may not be practical to plan a fully symmetrical building, any effort to reduce the eccentricity of the effective inertial force due to the noncoincidence of the centers of mass and of rigidity will pay off in reduced torsional stresses, which can be critical in corner columns and end walls. Locating the major stiffening elements near or along the plan periphery of a building substantially improves the torsional resistance of the structure.

3. Building multiple lines of defense--Cantilever walls can provide excellent resistance against lateral loads and can greatly reduce deflections. However, for seismic conditions they offer only a single line of defense. Should a large excitation require yielding, this is likely to cause permanent deformations near the base, and may lead to early misalignment in the building. Regular arrangements of openings in cantilever walls enable coupled walls to be formed. In seismic areas it is essential that the coupling beams rather than the walls form the weaker elements. With suitable detailing, coupled structural walls can be both efficient in load resistance and sufficiently ductile. Energy dissipation, when required, can be well dispersed over the height of the structure, and thus several lines of defense may be mobilized when extreme displacements are imposed on a building.

In general, a high degree of static indeterminacy is desirable in earthquake-resistant buildings. It is further desirable that an advantageous sequence in the propagation of yielding be established, so that damage in repairable and less critical areas will occur first and the principal gravity load carrying units will receive the greatest degree of protection. The designer must establish an intelligent hierarchy in the most probable strength levels which he intends to provide for each structural component.

In connection with the above, it is interesting to note the current approach to seismic design in New Zealand, as embodied in New Zealand Standard 4203:1976. This code requires that [39] buildings expected to undergo flexural ductile yielding be designed by a procedure called capacity design. In the capacity design of earthquake-resistant structures, energy-dissipating elements or mechanisms are chosen and suitably designed and detailed, and all other elements are then provided with sufficient reserve strength capacity to ensure that the chosen energy-dissipating mechanisms are maintained throughout the deformations that may occur.

4. Tying together of elements--The need to adequately tie together all the structural elements making up a building, or a portion of it which is intended to act as a unit, cannot be overemphasized. This applies to superstructure as well as foundation elements, particularly in buildings founded on relatively soft soil. Here, attention should be focused on the design of the segments of elements at and near the joints, since these are generally the regions which are most critically stressed.

Adequate connections should be provided across construction joints if they are required between parts of a building or between the main portion of a structure and an appendage, e.g., stairway enclosure, carport, etc..

5. Prevention of hammering--The different portions of a building should either be tied together adequately or separated from each other by a sufficient distance to prevent their hammering against each other.

Expansion or similar joints used to separate parts of a building which differ considerably in height, plan size, shape, or orientation should be sufficient to allow the components to sway independently of each other without impact. Any required passageway, corridor or bridge linking structurally separated parts of buildings should be so detailed as to allow free, unhindered movement during an earthquake.

In order to avoid hammering between adjoining buildings or separate portions of a building when vibrating out of phase of each other, a gap (perhaps filled with readily crushable material) equal to from four to six times the sum of the calculated lateral deflections of the two structures under the design (code) seismic forces, or the sum of the maximum deflections of the two structures as indicated by a dynamic analysis, would be desirable.

6. Infilled frames--The use of very stiff walls to fill the spaces in relatively flexible frames should be considered carefully during the preliminary design stage. The presence of rigid infill (having corresponding strength) causes the infilled frames to behave like cantilevers, thus totally changing the behavior of the frame elements.

If the infilling material is intended to act in combination with the enclosing frame, then it should be designed and constructed to ensure this composite action. Proper reinforcement and connection to the enclosing frame are essential. The analysis should likewise consider the increased stiffness and modified behavior of the infilled frames.

If the infill is made of fairly brittle material, such as glass or hollow brick masonry, and is not expected to contribute significantly to the lateral resistance of the frame, then it should be effectively isolated from the surrounding frame by gaps or readily crushable or yielding material to allow sufficient relative movement between the frame and such elements. The disastrous effect of deformation incompatibility between flexible frames and brittle infills has been observed in many earthquakes.

7. Reduction in the clear height of columns--The effect of introducing low walls between columns, as shown in Fig. 11, should be

noted. The reduction in height of the columns increases their stiffness with respect to bending in the plane of the wall. This will cause the columns to be subjected to greater horizontal shears than they would be expected to develop if the walls were absent. This is in addition to the effect which a decrease in the period of vibration of the structure--due to the increase in stiffness of the columns--will bring. The reduction in height also reduces the lateral deformation capacity of the columns in the plane of the wall. The use of such walls without allowing for their effects on the columns has been known to cause severe distress in portions of the columns above the wall.

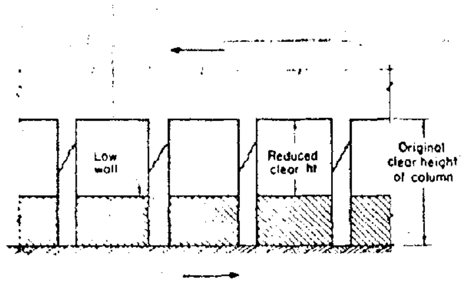


Fig. 11: Effect of Introducing Low Walls Between Columns

SEISMIC PERFORMANCE OF REINFORCED CONCRETE STRUCTURAL SYSTEMS

The performance of reinforced concrete buildings in many past earthquakes [40] has demonstrated that such buildings, when properly engineered, can withstand severe earthquakes not only without collapse, but also without serious damage to either structural or nonstructural elements. Collapse of reinforced concrete structures, as well as failure of individual structural elements or their connections, when such occurred, could be traced either to a lack of adequate strength and ductility, inadequate construction procedures, or to a lack of attention to the design considerations enumerated in the previous section. Damage to nonstructural elements, on the other hand, could be traced either to insufficient structural stiffness or to a lack of proper detailing of the connections between structural and nonstructural components.

While the foregoing, in general terms, affirms the adequacy of the current aseismic design procedures, serious deficiencies exist in the current capability for predicting the actual seismic performance of a structure. In the case of earthquake excitations, it is usually necessary to predict the force-displacement relationships for each story of a structure. The lateral displacement at any story (ΔH_i) can be expressed as a function of the gravity forces acting on the structure [G(t)], and the dynamic characteristics of the soil and the structure, which can be represented symbolically by the period [T(t)] and the damping coefficient [$\xi(t)$]. Thus [41] :

$$\Delta H_i(t) = f\,[G(t),\ T(t),\ \xi(t)] \qquad (1)$$

An analysis of the parameters involved in Eq. (1) indicates the following difficulties with the prediction of seismic response [41]:

1. All the parameters are time-dependent, except that the gravity forces usually remain practically constant for the duration of an earthquake. Thus, (a) the effect of the inertia forces developed at the masses cannot be neglected, (b) the rate of loading may be high enough to

affect the static-mechanical characteristics of the materials, on the basis of which the dynamic characteristics of the structure $[T(t), \xi(t)]$ are usually predicted, and (c) the possibility of low-cycle fatigue as well as incremental collapse must be considered.

2. The inertia forces depend not only on $U_g(t)$ but also on $\Delta H_i(t)$, $T(t)$, $\xi(t)$. This interaction between structural response and the forces themselves poses particularly intricate problems.

3. Lateral displacements, $\Delta H_i(t)$, depend on the ground motion occurring at the foundation of the building, rather than on the free-field ground motion, $U_g(t)$. The actual ground motion depends on soil-structure interaction. This interaction affects not only $\Delta H_i(t)$, but also $T(t)$ and $\xi(t)$.

4. Structural elements interact with one another and with nonstructural elements in a complex manner which depends on the detailing of their joints and connections.

5. Since inelastic deformations are not single-valued functions of stress, but are dependent upon the prior deformation history, a knowledge of both the critical loading combination and the history of loads is necessary.

The above difficulties are a clear indication of the need for experimental studies of actual buildings under real earthquakes. In fact, several reinforced concrete buildings and their surroundings around the world have been instrumented, and are currently under observation. Unfortunately, however, one cannot afford to wait for extreme earthquake excitations to occur in the vicinity of these few buildings to learn about their inelastic behavior. Ideally, the next best approach would be the testing of actual instrumented structures under simulated extreme excitations. However, the technical and economic problems associated with the generation of such extreme excitations have so far proved intractable. A more feasible approach is to reproduce ground motions by means of controllable shaking tables. Many small and a few medium-size simulator facilities are already in use. Although the research potential of these facilities is excellent, they can only test small-to-medium-scale models of complex structural systems. These models are usually inadequate to investigate in detail the actual dynamic characteristics and failure mechanism of the prototype. Furthermore, the reproduction of the actual ground motion (three components of displacement) is not easy. Most shaking tables reproduce only one of the three components at a time.

The foregoing indicates that at present the testing of actual complex structures under extreme dynamic excitations, real or simulated, is not altogether feasible. The most logical alternative is to subject actual structures or large-scale models to equivalent pseudo-static forces intended to induce effects similar to those of real dynamic excitations. Since in reality the inertia force at each concentrated mass varies with time, depending on the interaction between the real dynamic excitation and the dynamic characteristics of the building, the simulation of the actual inertia forces by simple static forces is a very difficult problem. One possible solution is to simulate what can be considered the critical combination of inertia forces that can be developed at a certain time. Rational selection of this critical combination requires integrated analytical and exerimental studies (e.g., the PCA investigation reported in [24,28,30]), because it will vary depending on what one is interested in studying. Further, even if a rational combination of inertia forces can be selected, the problem of how to vary the magnitude of these forces still remains, since the behavior of reinforced concrete is very sensitive to the loading path. This problem has usually been solved by adopting arbitrarily selected load sequences. In spite of these shortcomings, however, tests of structures and structural models under pseudo-static loading have yielded much valuable information.

Another approach which has proved useful, attempts to predict the response of a complete structural system from results obtained in studies of its structural elements. Concerning this approach, Newmark and Hall [42] have commented: "The strength of the combined system, the damping in it and the mode of failure can in some cases be inferred from the properties of the individual element; however, these members interact on one another in a complex way and in different ways for different types and directions of loading, and the interaction is a problem which must be taken into account in detail much more accurately than has been the case in the past if adequate lateral resistance to dynamic forces is to be achieved."

Bertero [41] has presented a thorough review of experimental studies that have been carried out on the behavior of reinforced, prestressed and partially prestressed concrete structures and their elements. Such studies will be analyzed under topics VII, VIII and X of this workshop. The prediction of earthquake performance of concrete structural systems need not, therefore, be discussed any further in this report.

PRESTRESSED (INCLUDING PRECAST) CONCRETE SYSTEMS

Prestressed concrete is seldom used in primary resistant structures against repeated loading conditions as severe as those expected to be caused by major earthquakes [9]. The principal reason for this has been the shortage of experimental evidence on the behavior of prestressed concrete members and member assemblies under such loading conditions. Prestressed elements, when utilized, have been used in conjunction with conventional frame, wall and frame-wall systems. Very often, only certain elements in the systems have been prestressed. Both cast-in-place and precast prestressed elements have been used. Precast wall and slab elements (mostly prestressed, occasionally conventionally reinforced) have been used in a structural system equivalent to, yet different in some respects from, the conventional cast-in-place wall construction--the so-called Large Panel structural system. A particular variation of this type of construction, the box-type structure, is fairly extensively used for low-rise buildings. Neither the available ductility nor the energy dissipating mechanism in this structural type has been fully explored as yet.

Prestressed concrete structures in general and precast structures in particular will be discussed under topics IX.1 and IX.2, respectively, of this workshop. Thus, the treatment here is limited to a few general remarks pointing out a number of features peculiar to prestressed (including precast) concrete.

Blakeley [43] has produced a comprehensive historical review of the seismic resistance of prestressed concrete structures and structural elements. According to him, most structures containing prestressed elements which have been subjected to earthquakes have performed well. Failures which have occurred appear to have been due mainly to failure of the supporting structure or of the connections. However, there is relatively little information on the behavior of fully framed prestressed concrete structures under strong earthquakes. The recent Romanian earthquake was the first in an area with a large number of precast structures (up to 9 stories high), and they performed well [44]. However, their response to this earthquake was mostly within the elastic range, making it difficult to reach an assessment as to their seismic resistance. For precast structures, the methods of joinery and their reserve strength and ductility present difficulties not always encountered in poured-in-place concrete [45]. These problems assume prime importance in the design of such structures.

Based on the experimental and analytical studies reviewed by him, Blakeley [43] has pointed out that:

1. Although the energy absorbed by a prestressed concrete member could be the same as or even larger than that absorbed by a similar reinforced concrete member, the greater elastic recovery of the prestressed member will result in a lower energy dissipation for cyclic loading. This is a drawback in seismic design. However, little is known of the energy-dissipation capacity of prestressed members under high-intensity cyclic loading. The energy dissipation would be greater for partially prestressed members once the mild steel yields, but the joints of such members present particular difficulties for precast construction.

2. Because of the lower energy dissipation capacity, and also because of the lower damping applicable to prestressed concrete relative to reinforced concrete, as observed in tests, a prestressed concrete structure is likely to suffer greater deformations or be called upon to resist higher forces under most strong earthquakes than a reinforced concrete structure of comparable mass and stiffness. A point in favor of prestressed concrete is that, to resist a given set of forces, a prestressed structure is normally considerably more flexible than its reinforced counterpart. This is a desirable feature for seismic resistance and partly counteracts the effect of the smaller energy dissipation under cyclic loading.

Spencer [46] studied the nonlinear dynamic responses to a strong earthquake (first 8 sec. of the N-S component of the 1940 El Centro record) of two reinforced and six prestressed concrete versions of a 20-story frame structure. An idealized bilinear moment-rotation hysteresis loop for the prestressed members was used. The prestressed structures were found to undergo higher lateral displacements and interstory drifts than the comparable reinforced concrete structures. However, the sectional ductility requirements of the prestressed structures were markedly lower. Studies directed toward confirmation and generalization of Spencer's observations would be most useful.

Further research is also needed in the following areas: damping tests of prestressed concrete structures; high intensity cyclic loading tests of prestressed concrete members and their connections.

CONCLUDING REMARKS

1. Experience in the earthquakes of the last 15 years has shown that both protection of human life and superior damage control can be attained in buildings stiffened by properly proportioned and detailed structural walls.

2. The incompatibility of flexible frames with brittle infills and finishes caused high economic damage to "nonstructural" building components in many earthquakes.

3. The effects of rigid elements on the seismic performance of structures make it imperative that proper account be taken of such elements in design. In reality, there are hardly any nonstructural elements, unless they are deliberately and carefully isolated from the structure itself. All elements attached to the structure and strained during the earthquake participate in the seismic resistance.

The discussion presented so far in this report points to the need for further research in a number of areas:

A. Integrated analytical and experimental research must be carried out in order to lay a basis for the safe and efficient design of the three basic framing systems - frames, structural walls and frame-wall systems.

Dynamic inelastic analyses are needed to arrive at reasonable estimates of the strength (with particular reference to shear) and deformability requirements in critical regions of the framing systems, corresponding to different combinations of significant structural and ground motion parameters. The ground motion parameters of concern are the intensity, the duration and the frequency characteristics of the excitation. The input motions to be used in dynamic analyses should be selected such that, for a particular intensity and duration, the frequency characteristics induce critical (near resonant) excitation in a structure in both the elastic and the post-yield stages of its response. This would normally require the use of a number of input motions in the analysis of the same structure.

Experiments are required to determine the minimum strength (again with particular reference to shear) and deformation capacities available in properly proportioned and detailed segments of the framing systems. The most promising experimental approach at the present time appears to be the testing of actual structures or large-scale models under equivalent pseudo-static forces intended to induce effects similar to those of real dynamic excitations. Particular attention must be paid to the selection of the appropriate combination and sequence of variation of the equivalent pseudo-static forces.

B. Experimental research is needed to establish force-deformation characteristics for all nonstructural elements incorporated into earthquake-resistant buildings, under appropriate repeatedly variable load combinations. On the basis of the data generated, rational limitations must be established on allowable interstory distortions. The allowable limits must be tied in some way to the intensity of the input motion.

Rational limits must also be established on the overall lateral deflection or drift. The prime consideration here is the stability of a structure under the action of gravity loads, when displacements are large.

The above research must be coupled with analytical studies employing dynamic inelastic analyses, leading to results such as those illustrated in Fig. 10. It would then be possible to translate drift and interstory drift limitations into allowable upper limits on the structural fundamental period (or lower limits on the overall flexural stiffness).

C. In order to establish the suitability of prestressed concrete for use in primary resistant structures against earthquakes, research needs to be carried out in the following areas: a) nonlinear seismic analyses of prestressed concrete structures, b) high-intensity cyclic loading tests of prestressed concrete members, their connections and subassemblages, and c) tests to establish differences in damping characteristics between reinforced and prestressed concrete.

D. Research should be directed to developing the full potential of the special devices and mechanisms which seek to reduce the forces induced in a structure by earthquake ground motions. Reliability studies in the form of earthquake simulator tests on large-scale models of these devices must play a prominent role in such research.

The above items are listed in the order of priorities the authors attach to the various research needs.

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STRUCTURAL SYSTEMS FOR EARTHQUAKE RESISTANT CONCRETE BUILDINGS

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DRAFT RECOMMENDATIONS

1. INTEGRATED ANALYTICAL AND EXPERIMENTAL RESEARCH MUST BE DIRECTED TO THE DEVELOPMENT OF RATIONAL AND PRACTICAL DESIGN PROCEDURES FOR THE THREE BASIC CONCRETE STRUCTURAL SYSTEMS--FRAMES, STRUCTURAL WALLS, AND FRAMES INTERACTING WITH WALLS.

    Reasonable estimates of the strength and deformability requirements in critical regions of the framing systems, corresponding to different combinations of significant structural and ground motion parameters, can only be established through inelastic dynamic analyses. Experiments need to be carried out to determine the minimum available strength and deformation capacities of properly proportioned and detailed segments of the structural systems. A rational design procedure must be based on correlation between analytical and experimental results.

2. RATIONAL LIMITS NEED TO BE ESTABLISHED ON THE ALLOWABLE INTERSTORY DISTORTIONS AS WELL AS ON THE OVERALL LATERAL DEFLECTION OR DRIFT OF A STRUCTURE, UNDER VARIOUS INTENSITIES OF SEISMIC GROUND MOTION.

    The limits on interstory distortions must be based on experimental data on the force-deformation characteristics of nonstructural elements incorporated into earthquake resistant buildings. The limits on drift must be based on considerations of the stability of a structure under gravity loads, when the displacements are large. Dynamic inelastic analyses must also be carried out, inorder to enable the drift and the interstory drift limitations to be converted into allowable upper limits on the structural fundamental period (or lower limits on the flexural stiffness).

3. FURTHER RESEARCH IS NECESSARY TO ESTABLISH THE SUITABILITY OF PRESTRESSED CONCRETE FOR USE IN PRIMARY RESISTANT STRUCTURES AGAINST EARTHQUAKES.

The following areas need particular attention: (a) nonlinear seismic analyses of prestressed concrete structures, (b) high-intensity cyclic loading tests of prestressed concrete members, their connections and subassemblages, and (c) tests to determine differences in damping characteristics between reinforced and prestressed concrete.

4. FURTHER STUDIES OUGHT TO BE AIMED AT REALIZING THE FULL POTENTIAL OF THE SPECIAL DEVICES AND MECHANISMS WHICH SEEK TO REDUCE THE SEISMICALLY INDUCED FORCES IN A STRUCTURE.

The main problem associated with the use of the above mechanisms concerns their reliability. Tests of large-scale models of the mechanisms using available earthquake simulator facilities appear to be the best possible way to conduct reliability studies at the present time.