IDENTIFICATION OF RESEARCH NEEDS FOR IMPROVING THE ASEISMIC DESIGN OF BUILDING STRUCTURES

V. V. Bertero*

1. INTRODUCTION

The purpose of this paper is to present and discuss problem areas that exist in the earthquake resistant (aseismic) design of building structures and to determine the most effective methods for obtaining the necessary information to resolve them. The identification of research needs is done by reviewing some of the general aspects involved in achieving an economical, serviceable and safe design for buildings located at sites with high seismic risk. These aspects, summarized in Table 1, may be classified in two groups: design and construction. For a more comprehensive list, see Ref. 1.

Because of space limitations, the present discussion will deal only with those factors related to design. This does not mean that construction is less important. On the contrary, the response of a building during any kind of excitation depends on how the building was actually constructed, and not on how the designer thought it would behave. Furthermore, design and construction are intimately related. The achievement of good workmanship depends, to a large degree, on the simplicity of the detailing of the members, connections and supports. This is especially true for reinforced concrete structures. In this case, it is possible on paper and even in laboratory specimens, to detail reinforcement so that considerable improvement can be achieved in seismic behaviour. Such design details, however, may be too elaborate to be economically feasible in the field.

Although the following discussion covers all of the design aspects listed in Table 1, emphasis is placed on the research needed for improving the establishment of design earthquakes, the selection of structural materials, the prediction of mechanical behaviour and the reliability analysis of members. A more comprehensive review of experimental studies needed concerning reinforced concrete structures is offered in Ref. 2. As will be discussed in more detail later, to achieve an efficient aseismic design, it is necessary to predict the mechanical behaviour of the structure under critical earthquake conditions. Building damage may result from different effects of an earthquake: (1) fault displacement, (2) strong ground vibration (shaking), (3) compaction and liquefaction of the soil foundation, (4) landslides, (5) tsunamis, and by other phenomena triggered by some of the above effects, such as fire. However, the critical condition that usually concerns the structural engineer, and is recognized by seismic design provisions in buildings codes, is the response of a structure to ground shaking caused by the transmission of earthquake vibrations from the ground to the structure and this is the only aspect that will be considered here.

The general problems involved in predicting seismic response of a building are symbolically defined and schematically illustrated in Fig. 1. As was stated above, the structural engineer is concerned with predicting the response (symbolically indicated by X_4 in Fig. 1), due to the shaking (vibration) of its foundation, X_3. As indicated in Fig. 1, the term, X_4, can be obtained by multiplying X_3 by a dynamic factor, D. Although this is a simple expression, the uncertainties involved in a realistic estimation of X_3 and D give rise to serious difficulties in obtaining an accurate numerical evaluation of X_4. A brief discussion of these difficulties follows.

Although it seems analytically feasible to predict the base rock motion at the given site (indicated by X_1 in Fig. 1), for an earthquake of specified magnitude, M, and focal distance, R_1, $[X_1 = f(R_1, M)]$, the prediction of X_3 must account for the effects of the soil layers underlying and/or surrounding a building. These effects can be classified in two groups: one is related to the influence of the dynamic characteristics of the different soil layers on transmission of X_1 to the free ground surface, which has been indicated in Fig. 1 by an attenuation or amplification factor, A; the other is due to the soil foundation-structural interaction effects, and has been symbolically represented by a factor, I. At present, large uncertainties exist regarding the realistic values of A and I, and major errors could be introduced by trying to quantify these two factors using suggested analytical techniques. It is clear that even if X_1 could be predicted with engineering accuracy, attempts to quantify the influence of soil conditions on X_1 to attain X_3 would result in a wide range of predicted values. Thus, the designer should not rely solely on results obtained from just one deterministic analysis. At least bounds of the possible variations in A and I should be considered.

The precise evaluation of X_4 at any point in the structure would require the establishment of its six components (three translational and three rotational); however, to simplify the discussion let us consider, as usually done, that the only significant components are the two horizontal ones, and that each of these components can be estimated independently. Usually, the lateral response

* Professor of Civil Engineering, University of California, Berkeley.

of the building, X_4, is defined by evaluating the total lateral displacement, ΔH_i, of each floor, Fig. 2. The prediction of the total lateral displacement of a particular building to a specific ground motion will depend upon the excitations acting on the structure and on the dynamic characteristics of the whole soil-structure system. In general, the main excitations acting on a structure during an extreme earthquake are due to: (1) gravity forces, $G(t)$, with the associated effects due to creep of the material, especially in the case of concrete structures; (2) changes in environmental conditions, $\Delta E(t)$, such as the stresses produced by the change in temperature; and (3) at least the three translational components of the foundation shaking, $X_3(t)$. The dynamic characteristics of the whole system, which change continuously as the structure is deformed into its inelastic range, might be summarized by representing them symbolically as the instantaneous: mass, $M(t)$; damping coefficient, $\xi(t)$; and the resistance function, $(R \text{ vs } \Delta H)_i(t)$, as is expressed in Eq. 1(a). As illustrated in Eq. 1(b), the dynamic characteristics can also be symbolically represented by the instantaneous; (4) fundamental period, $T(t)$; (5) damping coefficient, $\xi(t)$; (6) yielding strength, $Ry(t)$; and (7) capacity to absorb and dissipate energy, which can be represented by the instantaneous available ductility, $\mu(t)$, which is a function of $\Delta H_i(t)$. Thus, see Figs. 1(a) and 1(b) below.

Analysis of the parameters included in Eqs. 1(a) and (b) clearly indicate the magnitude of the difficulties of trying to predict response to earthquake ground motions. One difficulty arises from the fact that all these parameters are functions of time, although the gravity forces and changes in environmental conditions usually remain practically constant for the duration of an earthquake. We are, therefore, dealing with a dynamic problem in which it is necessary to consider two important effects of the time variation of the excitations and of the response: first, the effect of the inertial forces developed at the masses; and second, the rate of change in intensity of the strains with time (rate of loading or straining). This rate may be high enough to considerably affect the so-called static-mechanical characteristics of the materials on which the dynamic characteristics of the structure $[T(t), \xi(t), R_y(t), \text{ and } \mu(t)]$ are usually predicted.

Because the inertial forces depend not only on $X_3(t)$, but are also influenced by the behaviour of the structure $[\Delta H_i(t), T(t), \xi(t), R_y(t), \text{ and } \mu(t)$ and the presence of other excitations $[G(t) \text{ and } \Delta E(t)]$, it is clear that the interaction between the structural response and the inertial forces results in serious difficulties not only in the theoretical prediction of the response, but also in the development of any rational experimental investigation. As will be discussed later, the only efficient way to overcome these difficulties is through the close integration of analytical and experimental studies of the problem.

Another source of difficulty is that $\Delta H_i(t)$ depends on the response of the whole soil-building system rather than on the structural system alone. The soil-building interaction affects the so-called free-field ground motion, $X_2(t)$ (Fig. 1), which is the one usually measured. Furthermore, the response of the building depends to a considerable extent on the interaction between the structural elements and the so-called nonstructural components.

From the above discussion, it is clear that to carry out the efficient aseismic design of a building, it is necessary to predict its response. To do this, it is necessary to first establish the controlling (critical) ground motions, X_3, commonly referred to as "design earthquakes," and then to obtain information regarding the dynamic characteristics of the whole soil-structure system, i.e. $T(t)$, $\xi(t)$, $R_y(t)$, and $\mu(t)$. More specifically, it is necessary to know the actual excitation-deformation relationship or restoring force-characteristics of the whole system which include stiffness, strength, energy absorption and energy dissipation capacities (loosely defined as ductility), and the different sources of damping.

(2) ESTABLISHMENT OF DESIGN EARTHQUAKES

The general philosophy of earthquake resistant design has been well established; (1) to prevent nonstructural damage in minor earthquake ground shakings, which may frequently occur in the service life of the structure; (2) to prevent structural damage and minimize nonstructural damage in moderate earthquake ground shakings, which may occasionally occur; and (3) to avoid collapse or serious damage in severe earthquake ground shakings, which may rarely occur.

The main problems in implementing this philosophy for any given building are as follows: (1) establishing what constitutes minor, moderate and severe ground shaking at the building site and the probabilities of their occurrence during the service life of the structure; and (2) determining or

$$X_4 = \Delta H_i(t) = F\{[G(t), \Delta E(t), X_3(t)], [M(t), \xi(t), (R \text{ vs } \Delta H)_i(t)]\} \quad 1(a)$$

$$X_4 = \Delta H_i(t) = F\{\underbrace{[G(t), \Delta E(t), X_3(t)]}_{\text{Characteristics of Excitations}}, \underbrace{[T(t), \xi(t), R_y(t), \mu(t)]}_{\text{Dynamic Characteristics of Whole Soil-Structure System}}\} \quad 1(b)$$

quantitatively describing the corresponding earthquake ground motions that should be used as the excitation, X_3 (Fig. 1), at the foundation of the building or what can be called the "design earthquake".

2.1 Evaluation of Present Methods

In the past, the design earthquake has been specified in terms of a building code zone, a site intensity, or a site acceleration [3]. Furthermore, in aseismic design, it has been conventionally assumed that the only important effect on the structure comes from its base translation along one horizontal component of the ground motion. Although the building is analyzed for a similar motion along the perpendicular horizontal component, the structure is usually designed for the envelope, rather than for the combination of these two disturbances [4].

The ground motion actually has six components, three translational and three rotational. For sites located near active faults (or in general, near the source of earthquakes), each of the six components of the ground motions can be important. Furthermore, Rosenblueth [4] has shown that each of these six components can play a significant role in the overall response of a building, and prediction of response should be based on the consideration of these components acting simultaneously. There is an urgent need to obtain actual records of all the ground motion components For studying their effects on the response of buildings and for determining the minimum information required by the structural engineer in order to define "design earthquakes". Unfortunately, the information needed varies according to the limit states that would control the design of the structure. Therefore, we must establish at least two main cases, one in which the design is controlled by the service limit states, and the other, by the ultimate states. While in the first case, the structure should practically remain in its linear elastic range of behaviour to avoid functional failure, in the second case, inelastic behaviour up to the point of incipient dynamic collapse can be tolerated.

The specifications of a design earthquake by a site peak ground acceleration alone, is generally inadequate. From the available data regarding ground motion and building response, it is, at present, generally accepted that for buildings whose structure should remain essentially in the linear elastic range during their dynamic response, one of the best ways to quantitatively describe a design earthquake is by using an average or smooth response spectrum [5]. The best method of obtaining this response spectrum is by a statistical analysis of the linear elastic response spectra corresponding to earthquake records obtained at sites with similar soil conditions. Studies of this type conducted by Newmark, Blume and Kapur [6], show that the only basic data necessary to construct possible design response spectra are the peak acceleration, velocity, and dynamic or transient displacement of the ground shaking at the site of the structure. Therefore, the following questions must be answered: (1) what reasonably expectable types of earthquakes represent the most severe seismic hazard at the site? and (2) for these types of earthquakes, what ground motions are reasonably expectable at the site?

Although for certain sites, there are sufficient seismic and geological data for estimating the minor, moderate, and major expected earthquakes [7,8], in most of the cases, these data are unavailable. Even if it were possible to answer the first question, i.e. to have knowledge of (1) the seismic history of the fault zones located in the proximity of the site, (2) the seismic history of the tectonic province, and (3) the magnitude-fault length relations, as was pointed out earlier, there would still remain the problem of accurately predicting changes in the characteristics of the ground motion when it propagates from its origin to the foundation of the building via crust and surface layers.

Comparison of obtained linear elastic response spectrum derived for regions of high seismic hazard with present code regulations points out that it would be economically feasible to design all structures to resist a major earthquake without undergoing significant inelastic deformation. In these cases, safety against incipient dynamic collapse, rather than serviceability requirements, usually controls the design and it should therefore be based on a realistic inelastic model [9]. In addition, the design earthquake should include all the parameters of the critical ground motion that could affect such nonlinear behaviour. Procedures for construction inelastic response spectra for design have been suggested [10]. Assuming an elasto-perfectly plastic behaviour for the spring of the single degree-of-freedom system, pseudo-acceleration and displacement inelastic response spectra are constructed directly from the linear elastic response spectra by using factors based on an acceptable ductility displacement ratio. Caution should be exercised when using these inelastic response spectra for multi degree-of-freedom systems, especially when the hysteretic behaviour of the real structure can significantly differ from the assumed elasto-perfectly plastic idealization.

The validity of deriving inelastic design response spectra directly from linear elastic response spectra can be seriously questioned. The information needed and used for computing linear elastic response spectra, although necessary, is not sufficient for predicting inelastic dynamic response. It should be complemented with information on the duration of strong ground shaking and the number, sequence and characteristics of the large (especially long) acceleration pulses (large ground velocity increments) that can be expected. Repetition of large acceleration pulses can lead to the accumulation of inelastic straining which can induce any one of the combination of the two types of failure, i.e. low cyclic fatigue and incremental (crawling) collapse, illustrated in Fig. 2.

Furthermore, the vibration theory of one degree-of-freedom system shows us that the type of excitation that induces the maximum dynamic response in a linear elastic system is quite different from the type of excitation which is critical for an elasto-plastic system. In the case of a linear elastic system, the critical type of dynamic excitation is a periodic one with a frequency equal to that of the system, because in this case, the dynamic magnification factor, D, Furthermore, the vibration theory of one degree-of-freedom system shows us that the type of excitation that induces the maximum dynamic response in a linear elastic system is quite different from the type of excitation which is critical for an elasto-plastic system. In the case of a linear elastic system, the critical type of dynamic excitation is a periodic one with a frequency equal to that of the system, because in this case, the dynamic magnification factor, D, can reach a maximum value approximately equal to $\frac{1}{2\xi}$ due to an engineering resonance-like phenomenon. Thus, for values of ξ ranging from 2% to 10%, D can attain values ranging from 25 to 5. Although acceleration pulses are not usually critical in linear elastic response (because the largest value of the dynamic magnification, D, for an impulsive type of excitation is only 2), they can become critical for an inelastic system. This is particularly true for a structure having a hysteretic behaviour close to the linear elasto-perfectly plastic idealization with a yielding resistance equal to or less than that corresponding to the average or effective ground acceleration of the pulse, i.e., $R_y = M\ddot{X}_3$ pulse. In the case of elasto-plastic systems, the existence of periodic short acceleration pulses in the ground motion contributes only to building the response of the system up to its yielding level because once the system begins to yield, the phenomenon of engineering resonance is depressed since the energy dissipated through even small inelastic deformations is equivalent to very large values of ξ.

From the above discussion, it is clear that the amplification factors to be applied to the maximum ground accelerations to obtain the seismic linear elastic response of a structure are usually controlled by the engineering resonance phenomenon that is induced by a series of short acceleration pulses with the same periodicity as the structure. On the other hand, in the case of inelastic structures, while this series of short pulses can usually build the response up to yielding of the structure, considerably larger deformations can be induced by the presence of just one long pulse with an effective average acceleration equal to or just greater than that corresponding to the yielding strength of the structure. This observation raises serious doubts as to the validity of obtaining the inelastic response spectrum directly from the linear elastic one.

The above observations were confirmed during the analyses of damage that the Olive View Medical Center Buildings suffered during the 1971 San Fernando Earthquake (11). The record of this earthquake obtained at Pacoima Dam, as well as that computed at the base rock of the Dam (12) are shown in Fig. 3. Most of the structures of this medical complex failed or suffered large damage as a consequence of the early and long pulse whose peak acceleration, in the case of the recorded dam accelerogram, was about 0.6 g. The analyses show that the effects of the pulse with the maximum peak acceleration, i.e. about 1.25 g, was not as severe as the one with a peak acceleration of 0.6 g. Inspection of the velocity time-history corresponding to the Pacoima Dam Accelerogram, Fig. 3(a), shows that the long acceleration pulse corresponds to an incremental velocity of 155 cm/sec, while the pulse with the peak acceleration of 1.25 g corresponds to an incremental velocity of only 84 cm/sec. This clearly points out that for inelastic seismic response, what is important is the largest incremental velocity, rather than the largest peak acceleration of the ground motion.

It is hoped that from the above discussion it is clear that in establishing or selecting the design ground motions (design earthquakes), one should remember that the type of ground motion that is critical depends on the type of behaviour that is expected to control the response of the structure. To emphasize the importance of this interrelationship between ground motion and structural behaviour, the author and his associates have carried out a series of studies (13); the results obtained from a simple example are presented in Figs. 4-8. In the analysis of the failure of the Ambulance Parking Canopy of the Olive View Medical Center, whose cross section is shown in Fig. 4, it was found that because of uncertainties regarding the behaviour of the foundation and the real stiffness of the structural members, estimates of its natural period ranged between 0.09 and 0.34 sec. This structure was subjected to a series of studies under different actual earthquake records as well as under simple idealized ground accelerations. Figure 5 shows three of these simple accelerograms. Results presented in Figs. 6 and 7 permit comparison of the relative displacement time-histories for two different idealized mechanical behaviours of the structure of Fig. 4 (linear elastic and elasto-perfectly plastic) when subjected to ground motions 1 and 3 of Fig. 5. The yielding strength of the elasto-perfectly plastic hysteretic model was selected to be equal to Mg/3. A comparison of the main results obtained from Figs. 6 and 7 and presented in Fig. 8.

Analysis of the results presented in Figs. 6-8 not only confirms the observation made above, but also indicates the difficulty of finding simple relations that can be used to derive the inelastic deformations directly from results obtained assuming just linear elastic behaviour. Furthermore, it points out that if the ground motion can contain long acceleration pulses, it would be necessary to design the structure with a yielding strength somewhat larger than the largest effective or average acceleration of these long pulses. This need is accentuated when one considers the possibility that the ground motion can contain two or even more of these long pulses having the same acceleration sign. This type of motion can undoubtedly lead to an incremental collapse, Fig. 2.

Because of the uncertainties involved regarding prediction of the time-history of future earthquake ground motions, efforts should be devoted to establishing bounds for the different parameters needed to define these long pulses. There is an urgent need to establish the largest pulse, i.e. largest incremental velocity and associated average or effective acceleration, that can be developed for different soil conditions according to the mechanical characteristics of the soil. If this can be established, the structural designer will at least know the upper bound of the energy input that can be transmitted to the foundations of the structure and can design the structure accordingly.

Ambraseys [14] and Brune [15] have conducted some theoretical studies which enabled them to estimate an upper limit for near-fault peak velocity in the range of 100-150 cm/sec. Although very important, this information is insufficient. What is needed is the estimation of the maximum incremental velocity and the associated accelerations. Solution of this problem will require close co-operation between geologists, seismologists, soil engineers and structural designers. Integrated experimental and analytical studies should be carried out in this area.

In general, it can be concluded that only the continued accumulation of statistical evidence from actual earthquakes can lead to improved estimates of the extreme ground shaking occurring at the foundation of a building and, therefore, to the establishment of rational design earthquakes. Thus, the most important aid to improvement in this field is the continued installation of strong motion seismographs in extensive networks so that sufficient seismographic data may be recorded for a wide range of geological and soil conditions during future earthquakes of moderate and large magnitudes.

2.2 Use of Isolation Techniques

Because of the great uncertainties involved in predicting future seismic ground motions in general, and especially those corresponding to the most devastating earthquakes that may be expected during the service life of a building, one partial solution would be to control the occurrence of such earthquakes. This, however, does not seem feasible at present. Another attractive and promising solution would be to control the ground motion input at the building's foundation by mounting the structural system on an isolator. Although techniques based on this approach have been used extensively in the case of machine foundations, in the case of buildings, especially when they are tall and slender, a practical solution for controlling the earthquake energy input is more difficult. Several isolation techniques and mechanisms for controlling the excitation input to buildings have been suggested, mainly by Japanese investigators [16]. Analogous studies of isolation systems and vibration absorbers were conducted by Wirsching and Yao [17].

From the results of the studies that have been carried out, it appears that from a practical point of view, it is feasible to control the earthquake input ground motions to the foundations of low, rigid buildings by means of some of the mechanisms proposed by Japanese researchers, or by taking advantage of the strength capacity and energy dissipation characteristics of the surrounding soil. In the case of tall, slender buildings, the control (isolator) system must perform two functions. First, it must prevent the build up of unacceptably large accelerations which may occur as a consequence of an engineering resonance phenomenon in one of the higher modes of the building which is excited by the high frequency components of the ground motions. Second, it must prevent the development of large deformation in the building which may occur as a consequence of its fundamental mode having been excited by the low frequency component of the ground motion. These two functions may be achieved by using an isolator system in conjunction with dampers capable of supplying a high degree of energy dissipation. This technique has been applied by Japanese investigators in the aseismic design of the 200m tall, Yasuda-Kasai Building in Tokyo [18].

One of the main problems regarding the use of these mechanisms concerns their reliability. This can be assessed by testing proposed devices under conditions similar to those which may occur in the case of real earthquake ground motions. Use of earthquake simulators (shaking tables) appears to be essential in this type of study. Although it would be desirable to conduct tests on full-scale models of these devices, this is not possible with the presently available earthquake simulator facilities except for very 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.

Maintenance of these devices is another problem that should be carefully considered before they are applied in practice. These devices should be inspected, tested and easily replaced, if necessary, after each moderate or severe ground motion or wind storm.

Another technique for controlling the earthquake energy input to the structure was originally suggested in 1929 and involves the use of a flexible first story [19]. A similar, partial isolation technique was proposed by Fintel and Kahn in 1970 [20]. They proposed the use of a flexible and soft (relatively low yielding strength) first story. Engineers have found this technique to be less attractive, however, as a result of the damges incurred during the San Fernando earthquake of 1971 [21]. The study carried out by Chopra, et al. [22], has shown that the shear wave, which is transmitted into the structure through the ductile first story, can be amplified near the upper part of the building by a whiplash action. Before putting such techniques into practice, further analytical and experimental studies should be conducted on these partial isolation techniques, and reliable devices should be developed to prevent large deformations in the soft story and to absorb the whiplash effect on the upper stories.

LAYOUT OF STRUCTURE: SELECTION OF STRUCTURAL MATERIAL AND STRUCTURAL SYSTEM

An inspection of Eq. 1 clearly points out the importance of the structural layout to the entire design process. The inertia forces depend upon the mass (quantity and distribution), damping, and the structural characteristics themselves (stiffness, yielding, strength, maximum strength, and energy absorption and energy dissipation capacities). Therefore, decisions made regarding the choice of layout for the structure and the choice of material (for structural as well as nonstructural elements) must play a significant role in the seismic performance of the structure during its lifetime. Problems related to the selection of structural materials and the structural system will be discussed separately; however, it must be recognized that the two are intimately related.

3.1 Structural Material

The material should be light and possess both a high degree of internal damping and excellent dynamic mechanical characteristics, i.e. it should be stiff, strong, and possess large ductility (large energy absorption and energy dissipation capacities). In order to effect an intelligent selection, it is necessary to have a good knowledge of the mechanical behaviour of available structural materials under the dynamic conditions imposed by the response of a structure to all levels of dynamic earthquake excitations.

For many years it has been recognized that the behaviour of materials under dynamic conditions is vastly different from their behaviour under static conditions (23-25). This is particularly true in three respects. First, in dynamic loading, the normal static stress-strain relationship is altered, permitting different deformational, energy absorption and energy dissipation capacities. In general, the mechanical characteristics of materials tend to improve with the increasing rate of load application. Second, dynamic loading may alter the mode of failure. In general, it enhances brittle failure; thus, the interaction of, and influence on, the fractural tendency of such conditions as severe restraints, residual stresses, discontinuities, flaws, thicknesses of materials and joints, and transfers of stresses from one material to another in composite materials, must be studied. Third, dynamic loading can cause failure by fatigue; low-cyclic fatigue being of special interest in this discussion.

As will be discussed later, the data available from the behaviour of structural components subjected to strain rates similar to those expected in real earthquakes appear to indicate that their effects could be neglected. The data, however, are scarce and, in general, comprehensive data on the behaviour of actual members and structural components under low-cyclic fatigue loading conditions at fast strain rates are still lacking. For example, in the case of reinforced concrete structures, reliable data should be obtained regarding the influence of the rate of strain on: (1) the stress-strain relationship of reinforcing steel and concrete, including tension, compression and shear; (2) the internal damping of the two constituent materials; (3) the stress transfer or bond between the two materials; and (4) the low-cyclic fatigue characteristics of each of the materials and their composite actions in different structural elements and assemblies, as well as how the detailing of reinforcement (including different types of anchorage, different degrees of confinement, etc.) can affect this behaviour, with special attention to the buckling of the main reinforcing bars.

In a detailed evaluation of the mechanical characteristics of materials of selected buildings at the Olive View Medical Center (26), these characteristics were found to exhibit considerable variations. For example, Figs. 9 and 10 show the frequency distribution diagrams of the field control test data for the two mixes of normal weight concrete used in the Main Treatment Facility (MTF) building. Figures 11 and 12 show similar diagrams for the A-15 steel bars mill test data. A summary of statistical evaluation from mill test data is shown in Table 2. In Table 3, comparision of observed and calculated values of the modulus of elasticity for concrete on compression is presented. The observed values were obtained from laboratory tests conducted on specimens cored at the site.

In view of the variability of the actual mechanical characteristics of presently available structural materials, present seismic code provisions specifying only the minimum and maximum strengths of the materials and recommending that the capacity of members and their design be based on these code specified strengths alone, are unreliable and can lead to unsafe designs, especially in the design of connections (anchorage) and against the effect of shear, particularly in the case of reinforced concrete structures (26-28).

There is an urgent need to develop more reliable materials, particularly those used in reinforced concrete, and for more stringent code specifications. As a consequence of the results obtained in the study reported in Ref. 26, specific recommendations have been formulated for changes in various code sections dealing with the design material characteristics and with the quality control requirements for structures subjected to seismic loading.

The general trend that started in the U.S. during the 1960's to fabricate and use both structural and reinforcing steel with increasingly higher yield strengths is undesirable from the point of view of achieving ductility. This usage is unfortunate because the attainment of large ductility capacities is perhaps the most efficient way of overcoming the many uncertainties presently involved in aseismic construction.

In the case of reinforced concrete construction, the use of lightweight aggregates is also becoming more common in the U.S. because of the increasing costs of producing and transporting normal weight aggregates. Furthermore, its use is very attractive for aseismic construction, due to the considerable reduction in weight and, therefore, mass. Unfortunately, because of its lower modulus of elasticity, very high compressive strength concrete mixes have been used in several cases to achieve a higher degree of stiffness. Recent studies carried out by Bresler and Bertero (29) regarding the behaviour of confined and unconfined concrete with different types of aggregates, have shown that as the compressive strength of some unconfined lightweight aggregate concretes is increased to beyond 3000 psi, they become considerably more brittle (Fig. 13). Confinement of concrete with all types of aggregate tested, was effective in developing large deformability. However, the effectiveness of concrete confinement in the performance of earthquake resistant reinforced concrete structures should not be based only on the extent to which the deformability is increased, but also on the ability of the confined concrete to sustain large deformations without loss of strength. Therefore, confinement should also increase the compressive strength of the concrete, so that it is possible to offset the loss of strength due to the reduction of the cross section resulting from crushing and spalling of the concrete cover.

Some of the results obtained in the study presented in Ref. 29 are illustrated in Fig. 14. These results show that for different concretes, the above two conditions of increased deformability and compressive strength are satisfied to a varying extent, and the effectiveness of confinement is highly sensitive to the type of aggregate used. The effectiveness of confinement can be characterized by two material constants, k_o and k_u, which are defined by relating the increased compressive strength, f_c, with the confinement pressure, f_r.

The maximum compression strength, $f^*_{c\ max}$, occurs after some strain ε^*_o and can be defined as follows:

$$f^*_{c\ max} = f_c + k_o f_r \qquad (2)$$

where f_c is the compressive strength of the same concrete, but unconfined. At very large deformations, $\varepsilon^*_u >> \varepsilon^*_o$, the compressive strength usually decreases to a value of f^*_{cu}, and can be defined as follows:

$$f^*_{cu} = f_c + k_u f_r \qquad (3)$$

The confinement pressure, f_r, depends on the geometric and material characteristics of the spiral wire, and can be expressed as follows:

$$f_r = \frac{2A_{sp} f_s}{D_c s} \qquad (4)$$

Assuming that the ductile spiral wire yields when the longitudinal strain in the concrete is in the range, ε^*_o to ε^*_u, and that the strain-hardening of the spiral is negligible in the range of these concrete strains, f_s is equal to f_y, and f_r can be calculated for given values of A_{sp}, D_c, and S from Eq. 4; values of k_o and k_u can then be calculated from Eqs. 2 and 3 using the test results. These values for the five different concretes used in the study are shown in Table 4. Early investigators have shown that the confinement effectiveness coefficient, k, varies with lateral pressure intensity and with longitudinal strain. However, in developing the ACI criterion for spiral reinforcement (Sec. 10.9.2 of ACI 318-71) and similar criteria which are based on the confinement of concrete, a constant value of k, usually taken as 4.0 to 4.1, has been assumed.

As shown in Table 4, the values of k for normal weight aggregate concrete vary in the range of from 0 to 7.0. For the two lateral pressures ($0.13f_c$ and $0.32f_c$), values of k_o at maximum compression are 7.0 and 5.0, respectively, and values of k_u at ultimate strength are 0 and 3.1, respectively. Based on these values, and noting from Fig. 14 that concrete behaves in a relatively ductile manner throughout a significant range of strains, a constant value of k = 4.0 may be justified for concretes such as E-5, particularly in the case of $f_r = 0.32(f_c)_{10}$.

For concretes B-3, B-5, R-3 and R-5, the values of k vary in the range of -1.0 to 4.4. Negative values of k_u indicate that compressive failure in the confined concrete may occur at values below the compressive strength of unconfined concrete. For the two lateral pressures $[f_r \simeq 0.1(f_c)_{10}$ and $f_r \simeq 0.3(f_c)_{10}]$, values for k_o at maximum compression range from 1.0 to 4.4 and values for k_u at ultimate range from -1.0 to 2.1. Based on these results for aggregates similar to those used in this investigation, a value of k in the range of 1.0 to 2.0 should be taken in developing design criteria based on the increase in strength due to the confinement of lightweight concrete. Therefore, in such cases the amount of spiral steel required in a column of lightweight aggregate concrete will be 2 to 4 times as great as that currently prescribed by the ACI Code. Because of the geometric limitations introduced by the size of the spiral wire and the minimum spacing, it would be virtually impossible to produce a spiral which would also allow normal placing of concrete.

The effect of the variable coefficient, k, is illustrated in Fig. 15. In this figure, the loss of the axial load carrying capacity for spirally reinforced concrete columns due to spalling is plotted against k, assuming that the spiral reinforcement was designed in accordance with the ACI criterion. This loss of capacity is expressed as a ratio and derived as

$$\text{Loss} = 0.85f'_c(A_g - A_c) - kf_r A_c ,$$

and using Eq. 4

$$\text{Loss} = 0.85f'_c(A_g - A_c) - 0.5k\rho_s f_s A_c \qquad (5)$$

According to the ACI criterion, $\rho_s = 0.425 [(A_g/A_c)-1](f'_c/f_s)$. By substituting this equation into the above, and dividing by $0.85f'_c A_g$, the following ratio is obtained

$$\frac{\text{Loss}}{0.85f'_c A_g} = (1 - \frac{A_c}{A_g}) - 0.25k(1 - \frac{A_c}{A_g}) \qquad (6)$$

Typical values of A_c/A_g for spirally reinforced square columns, vary from approximately 0.4 to 0.6; for round columns this ratio varies from approximately 0.5 to 0.7. The loss ratio for typical values of A_c/A_g is plotted in Fig. 15, which clearly illustrates the significant losses that can occur due to k values lower than 4.

Because recent suggestions for improved design of earthquake resistant reinforced concrete structures rely on the beneficial effects of confinement on concrete behaviour, a brief discussion of the implications of the conclusions of the study reported in Ref. 29, are presented with regards to the prediction of seismic behaviour.

1. Deformation characteristics of confined concrete are sensitive to type of aggregate and to relative amount of confining pressure. The modulus of elasticity of concrete in compression varies not only with compressive strength and unit weight, but also with the type of aggregate used. Prediction of modulus of elasticity using the ACI formula may significantly overestimate modulus values of confined concrete, and therefore estimations of natural periods T of reinforced concrete structures can be affected. This effect should be considered in seismic analysis by allowing for corresponding variations in estimated values of T.

2. Confinement of concrete, with all types of aggregates, is effective in developing large deformability, i.e. large ultimate strains. This characteristic is the major factor in the improved performance of elements with spirally confined concrete as it compensates for some of the losses in strength and stiffness of concrete under cyclic loading.

3. The increase in compressive strength due to confinement is about twice as great for normal weight concrete than for lightweight concrete. Therefore, caution should be used in applying equations derived from results obtained using normal weight aggregate concrete to predict behaviour of lightweight concrete.

4. The low effectiveness of confinement in some concretes may lead to significant losses in compression capacity when spalling occurs in reinforced concrete elements. This is of the utmost importance in the case of seismic design of column elements since these elements should at all times be able to resist the effects of gravity loads and overturning moments.

5. Increasing the rate of strain increases the compressive strength and stiffness modulus over those observed under a slow rate of loading. The increase in modulus, however, is relatively smaller than the increase in strength, which can be about 20% under a strain rate of $\dot{\varepsilon} = 10{,}000 \times 10^{-6} \frac{\text{in./in.}}{\text{sec.}}$.

6. Cyclic loading at high strain rates with peak stresses in the range of static compressive strength may produce significant reduction in post-cycling stiffness and strength. Therefore, possible effects of the observed reduction of deterioration in energy absorption and energy dissipation capacities, as well as in the shear strength and bond characteristics of confined concrete in structural elements subjected to severe cyclic loading, should be investigated.

From the above discussion, it can be concluded that:

(1) The real dynamic mechanical characteristics of materials can vary over a wide range. This variation, together with uncertainties regarding workmanship and quality control in the field, should be considered in the design process. Since design cannot rationally be based on one deterministic analysis using specific values, a nondeterministic approach where at least bounds on the range over which the main mechanical characteristics of the materials as used in the field can vary should be used.

(2) The above variations and uncertainties should be minimized. On the one hand, variations in mechanical characteristics of materials can be lessened through closer cooperation between material producers, researchers, and designers aimed at the production and use of more reliable materials. Closer cooperation between designers, constructors, inspectors, and building officials can, on the other hand, ensure effective compliance with standards established by plans, specifications, and building regulations.

3.2 Structural System

Building structures may be of many types and configurations. Because the earthquake resistance characteristics of most recently suggested structural systems have not yet been fully defined, integrated experimental and analytical studies of the seismic behaviour of these different systems are needed. In discussing the research needs in this area, it is convenient to establish two general groups of structural systems: (1) those used in non-engineered buildings which can be defined as "Non-Engineered Structures", and (2) those used in engineered buildings which can be termed, "Engineered Structures".

3.2.1 Non-engineered Structures

Most places of residence -- homes, lodging houses -- and small, low-rise buildings are non-engineered structures. Within the last decade, new construction systems for these types of buildings have been introduced. Most of them use prefabrication techniques. The overall seismic behaviour of these types of buildings should be studied, with an emphasis on the anchorage of wall and other components to the floor and roof. To carry out this study, it will be necessary to develop large-scale facilities which would permit the testing of the building in full-scale. Although the ideal would be to test these buildings in earthquake simulators, the development of a large-scale pseudo-static testing facility could also contribute to improvements in the construction of these types of buildings, especially in those structures where, owing to their arrangement, it would be possible to approximately simulate the effects of ground motion excitations by pseudo-static forces.

3.2.2 Engineering Structures

Buildings having a distinct structural system fall into this group. Although some guidelines have been offered [30] for the selection of the most efficient structural system, these were based on a few analytical studies rather than on the real behaviour of the systems under earthquake excitations. There is an urgent need for integrated experimental and analytical studies of the earthquake resistant design of new structural systems that have been proposed and are already in use (e.g. suspended systems, staggered truss (wall) systems, tubular and bundle tube systems, etc.). Because most of these new systems are used in tall buildings, it will not be economically feasible to study their effectiveness by testing full- or even large-scale models. Instead, model studies should be carried out using medium-scale shaking tables.

Another approach for developing better earthquake resistant structures is to control the dynamic response of the building by means of dampers (energy absorbing devices) (31). Although the development and testing of new damping devices could be carried out with the presently available testing facilities, the final check of the reliability of these devices in controlling the response of the structure could be obtained only through tests of the largest possible full-scale model of the building in which they would be installed. As in the case of isolator mechanisms, problems of maintenance, replacement, etc., should also be carefully studied before adopting such devices.

Until more reliable information regarding the behaviour of new systems is available, it is recommended that designers continue using systems whose performance during severe earthquakes and/or through comprehensive experimental and analytical studies have proven their earthquake resistant efficiency. In selecting these structural systems, it is important for designers to recognize that the dynamic forces to be developed in a building during earthquake ground motions can be controlled by the proper selection of its structural system and by the amount and distribution of the building masses, as is shown by the inter-relationship in Eq. 1. The need to select very regular forms for the layout of buildings and structural systems having symmetrical distributions of the masses and stiffnesses and strengths for minimizing the undesirable significant torsional response is something that cannot be overemphasized. Furthermore, it is necessary to give the building the means for finding its way out from critical periods of its probable seismic dynamic response without failing. This can be accomplished by developing combined structural systems which offer several defense lines (the high redundancy of structural system components) for resisting the effects of severe ground shaking. Not only should each of these defense lines be ductile, but they should also be coupled with very ductile elements. To illustrate the importance of the above points, several examples are briefly discussed.

3.2.2.1 Main Building of Olive View Medical Center (11,21)

This was a six-story building. An examination of the layout of this building, Fig. 16, reveals the use of large unnecessary masses (about 1 1/2 ft. of dirt on the first floor). The structural system also has significant discontinuities. While the upper four stories consisted of shear walls combined with moment resisting space frames, the lower two stories had only a moment resisting space frame system. The floor system consisted primarily of a flat slab-column system with drop panels at the columns. Tied and spirally reinforced concrete columns were used. The shape and reinforcement of these columns differed from story to story, as is illustrated in Fig. 17.

The combination of discontinuities and the use of unnecessary masses played an important role in the performance of the building during the San Fernando earthquake of February 1971. Although the main building did not collapse, it was, from the functional point of view, a complete failure, despite the fact that seismic resistance coefficients for the ground and first story were estimated as 0.30 and 0.44, respectively, i.e. considerably higher than those required by the code provisions (21). The permanent deformations of the first two stories were so large (up to a 30 in. relative displacement between the first and second floors was measured) that the structural and non-structural damage was beyond economical repair.

3.2.2.2 Banco de America and Banco Central of Managua (32,33)

Studies of the earthquake damage during the Managua earthquake of December 23, 1972, indicated that while buildings with properly detailed and constructed moment-resisting frames could sustain large seismic excitations without collapse, several buildings developed deformations large enough to endanger life. On the other hand, buildings with properly designed and constructed reinforced concrete shear walls performed exceptionally well. The more symmetrical the plan of the building and the greater the ratio of shear wall area to gross floor area, the better the performance of the building. A good example of these findings can be offered by comparing the performance of the two banks illustrated in the photographs of Fig. 18. The Banco de America generally performed very well, although the structure suffered some structural and nonstructural damage. Its excellent performance can be attributed to the symmetry and uniformity of the distribution of the masses and structural hysteresis throughout the building (Figs. 19 and 20).

The structural system, which can be considered as a combination of coupled ductile walls with a framed tube, appears to be an excellent system for aseismic design, providing several lines of defense by where the behaviour of the whole system can accommodate the demands of different kinds of severe earthquake ground motions. The fifteen-story tower of the Banco Central suffered considerable structural and non-structural damages despite the fact that its design was carried out for lateral loading considerably in excess of any buildng code requirements in the United States at the time of its construction in 1961. The damage can be attributed mainly to the flexibility of the structural system. The lateral resistance was offered primarily by frame action. As a consequence of the very flexible floor system (Fig. 21), relatively large lateral story displacements and vertical floor movements took place. The stiffer reinforced concrete walls around the elevators, together with the masonry infill wall that was used on the west side (Figs. 18 and 21), introduced an extremely large torsional eccentricity into the building that contributed to the observed damage. It should also be noted that there is a discontinuity in the structural system at the fourth floor level where the closely spaced columns shown in Fig. 21(b) terminate at transfer girders. These girders are supported by only 10 columns as is shown in Fig. 21(a). Irregular forms resembling the one used in this building should be avoided whenever possible.

3.2.2.3 Deep Spandrel Girder - Short Column Systems

An architectural layout which has been commonly used in the U.S. for educational facilities, hospitals, medium-rise office buildings and parking garages is one employing deep spandrel girders and short columns, as illustrated in Fig. 22. The fact that buildings using this type of structural system suffered heavy damage during the 1964 Alaska, 1968 Tockachi-Oki, 1971 San Fernando and 1972 Managua Earthquakes indicates the undesirability of this type of structural form. Because of the short clear height of the columns, high shear forces develop in these elements, usually resulting in very brittle types of failure. Therefore, it is necessary to either abandon this type of construction or, at least, to investigate practical means for increasing the ductility of the resulting short columns. This last alternative is presently being pursued at the University of California, Berkeley [34].

It is hoped that the above examples sufficiently illustrate the importance of recognizing the significant role that the general architectural layout and the selection of the structural system play in the overall seismic performance of the building. The best policy in earthquake resistant design is to avoid problems whose solutions are unreliable. Close cooperation between the architect and structural designer established at the beginning of a project can help in carrying out such a policy.

4. PREDICTION OF MECHANICAL BEHAVIOUR OF STRUCTURES

This aspect of the design procedure involves two fundamental steps: (1) modelling of the structure and (2) design method.

4.1 Modelling of the Structure

Realistic modelling of structural characteristics is of the utmost importance in achieving an efficient earthquake-resistant design, and requires the accurate representation of the behaviour of the constructed building, particularly its masses, support conditions, connections, element stiffnesses and strengths, and foundations. The possible interaction of nonstructural components with structural elements should also be considered. Decisions regarding the representation of these characteristics require a clear understanding of the expected behaviour of the structure, an understanding which can be gained through integrated analytical and experimental studies. As will be discussed later, experimental data are needed to gain the required knowledge of structural behaviour, especially in the case of severe earthquakes where the structure may undergo large inelastic deformations. For example, the accurate representation of the foundation supports requires knowledge of the interaction between the structure and soil. Furthermore, since the nature of this interaction can change during a severe earthquake ground motion as a consequence of the change in the mechanical characteristics of the structure and/or soil with intensity of straining, the designer should be aware of these possible changes and, at least, estimate their effect on the behaviour of the selected model.

Because of uncertainties regarding the different factors that should be considered in the modelling of real structures, the designer must realize from the beginning that the design cannot be based on just one deterministic analysis of a selected model. The designer should consider several models based on the possible combinations of the bounds of the ranges over which the different parameters governing the behaviour of the real structures can vary. To illustrate this point, let us re-examine the apparently simple case of the Ambulance Parking Canopy of the Olive View Medical Center [11,21].

The structure of this canopy consisted of a relatively heavy concrete roof slab (433 kips) supported by 12 stiff, reinforced concrete tied columns, as illustrated in Fig. 4. The north columns were sheared off at the top, with the lower portions remaining practically vertical. The permanent roof displacement, shown in Fig. 23, occurred after the failure of all the north columns. The design of this structure was based on the model shown in Fig. 24, i.e. assuming a hinge and support at the top of the footing. Based on this model, the computed shears in the columns were so small (Fig. 24) that shear reinforcement was not required. A more careful analysis of the structure's building plan would have led to the necessary consideration of the restraint offered by the ground slab which was 2 ft. above the level of the columns foundation. The manner in which this slab was constructed restrained the column, causing a nearly fixed-fixed condition to develop for the portion of the column above the slab; this development more than doubled the value of the calculated ultimate shear in the column (Fig. 25). This inaccuracy in modelling which resulted in a considerable underestimation of the required shear reinforcement, was the most important factor in the failure of the canopy.

The consideration of possible bounds in the restraints offered by the foundation and ground slab would also have shown that the natural periods of this structure (the estimation of which is essential to aseismic design) could vary between 0.09 sec. to 0.34 sec. This wide range of variation re-emphasizes the need for using nondeterministic concepts in the aseismic design of structures.

4.2 Design Method

Selection of an appropriate design method requires the establishment of design criteria and its application requires the following: (1) modelling of the structure, (2) structural and stress analysis, and (3) proportioning and detailing of members and their supports and connections. To accomplish these three steps, it is necessary to predict the mechanical behaviour of the structure under critical earthquake ground motions (design earthquakes). Assuming that the critical ground motions have been established, the first problem that arises is in determining the information necessary for predicting such mechanical behaviour. To do so, we must first distinguish between the gross proportioning of members and their actual detailing.

For the gross proportioning of members, it is necessary to carry out structural analysis which usually requires information on the lateral force-displacement relationship for each story of the building (Fig. 2). On the other hand, for detailing, it would be necessary to obtain the required moment-rotation and/or shear force-distortion at the different regions of each member and their connections and/or supports undergoing inelastic deformations. The locations of possible overstressed regions are illustrated in Fig. 26 for one floor and are referred to as "critical regions".

In the prediction of the force-displacement relationship for each story of the building, serious difficulties are encountered because of the interaction problems that have been mentioned in the Introduction where Eq. 1 was discussed. These interaction problems indicate the need for the following studies:

4.2.1 Studies on Behaviour of Actual Buildings

4.2.1.1 Under Real Earthquake Ground Motions

Although several buildings and their surroundings, mainly in Japan, have been thoroughly instrumented and observed for several years (35,36) and some significant data have been obtained regarding their behaviour |mainly on T(t) and $\xi(t)$|, this information pertains only to minor or service level earthquake excitations. Some significant data on the hysteretic behaviour of a nine-story, reinforced concrete building during the San Fernando Earthquake of 1971, was reported by Iemura and Jennings (37). In spite of the fact that insignificant yieldings occurred, the period of vibration of the structure increased by about 50% (from 0.66 to 1.0 sec). Data have not yet been obtained under earthquake shaking severe enough to induce significant inelastic deformations of the buildings. Such data are needed. Therefore actual buildings -- with different dynamic characteristics -- and their surroundings (soil) located in all the seismic areas of the world should be thoroughly instrumented. The instrumentation of buildings and surroundings should be planned to obtain sufficient data for separating the effects of each of the six components of the ground motions. Efforts should be devoted to obtaining data regarding actual soil-structure interaction for the possibility of reconciling the considerable differences between damages that are expected from estimated values of overturning moments and the actual damages that have been observed after severe earthquakes. This in turn requires diligence in developing reliable measurement transducers and recording devices whose costs -- initial installation as well as maintenance -- would be sufficiently low to permit large-scale use.

4.2.1.2 Under Simulated Earthquake Excitations

Because the probability that any of the instrumented buildings will be subjected to severe ground motions due to a real earthquake is very small, it is necessary to supplement the above sources of information by trying to generate an earthquake-like environment by means of controllable sources. The use of underground explosions seems most promising. Some studies have already been carried out using explosions, but most of the building response data obtained have been in the elastic range. A recent study has shown that by sequentially firing underground nuclear explosives (yields of approximately 10 and 200 kilotons), it is possible to produce an earthquake-like motion sufficient for subjecting buildings to severe inelastic deformations (38). Further studies should therefore be made of this possibility.

Conceptually, an important way of supplementing the information from field instrumentation is by using shaking tables large enough to permit the application of arbitrary ground motions to full-scale buildings constructed on them. However, present construction of such tables does not seem feasible, chiefly because even the largest table whose feasibility study has been carried out at present (100 ft x 100 ft) permits the testing of only three- or four-story buildings at full-scale without the surrounding foundation material (39).

From the above considerations, reliable information regarding the real response of buildings to severe earthquake ground shaking cannot be expected in the very near future. Thus, other ways of obtaining the needed information should be investigated. One possibility is to test small-scale models of buildings on medium shaking tables. However, the dynamic testing of models in their nonlinear range in compliance with the requirements imposed by the laws of dimensional similarity is difficult. Furthermore, carrying out comprehensive studies using dynamic testing would require very many tests including not only a wide class of probable ground motions, but also variations in the structural parameters controlling the building's response. Thus, the cost involved would probably prohibit such a comprehensive study.

Dynamic earthquake testing has the basic disadvantage that input motion is over in a few seconds, and the probability that the malfunction of the devices controlling the motion input and/or the recording instruments is usually high because of their complexity. When these difficulties are coupled with the fact that these tests usually damage and sometimes destroy the model, it is clear that it would be convenient to replace the dynamic excitations by equivalent pseudo-static excitations.

4.2.1.3 Under Equivalent Pseudo-static Forces

The main advantage of experimentally studying the behaviour of buildings under pseudo-static excitations is that the test can be stopped at any time to check the instrumentation, recording devices, state of the specimen's damage, etc. Thus, it is possible to change the program of excitations to be applied according to the data and results obtained during the tests. Furthermore, since it is possible to observe the sequence of damage, a better picture of the actual behaviour of the structure can be obtained.

This testing method has, however, certain limitations. The time effects are apparently eliminated, i.e. inertia forces are replaced by equivalent pseudo-static loads, $H_{(h)}$, and by applying these loads slowly enough, effects of the rate of straining and $\xi(t)$ become negligible. Thus, the lateral displacement, ΔH_i in Fig. 2, can be expressed as a function of the excitation histories applied to the structures and of the pseudo-static mechanical characteristics of the structure.

$$\Delta H_i = f\Big(\underbrace{\left[G_{(h)},\ \Delta E_{(h)},\ H_{(h)}\right]}_{\text{Excitation Histories}},\ \underbrace{\left[K_{(\Delta_H)},\ F_{(\Delta_H)}\right]}_{\text{Pseudo-Static Mechanical Characteristics}} \Big) \qquad (7)$$

where in

$H_{(h)}$ = Lateral Forces, magnitude and history.

$K_{(\Delta_H)}$ = Stiffness of the structure which in the inelastic range varies with the total deformation history.

$K_{(\Delta_H)}$ = Foundation deformation effects which vary with the total deformation history.

It should be noted that although it would have been possible to include the foundation deformation effects directly in the $K_{(\Delta H)}$ by defining this as the stiffness of the whole soil-structure system, it has been found convenient to separate them. Reasons for this separation will become clear when the effects of the strain rate are discussed.

(i) Strain Rate, $\dot{\varepsilon}$, Effects. -In discussing structural materials, it has been pointed out that the behaviour of materials under dynamic conditions may be vastly different from that under static conditions. Therefore, it would seem that when the actual earthquake excitations are replaced by the effects of equivalent pseudo-static forces, a severe limitation is introduced. However, available results regarding the possible effects from the rate of straining that can be induced during the response of real structures to severe earthquake ground motions (40) indicate that:

1. The stiffness of the structure itself increases with increased $\dot{\varepsilon}$. For the maximum $\dot{\varepsilon}$ expected, the dynamic secant stiffness at first yielding may be increased not more than 10% over the static one.

2. The strength increases with increased $\dot{\varepsilon}$. This effect, however, diminishes with increasing strain amplitude. In the case of reinforced concrete, the dynamic cracking, as well as the first yielding strengths, can be increased about 25% over the static one. On the other hand, the maximum and ultimate strengths do not appear to be significantly affected.

3. The mode of failure for ductile structures is unaffected by $\dot{\varepsilon}$.

4. The energy absorption and energy dissipation capacities (ductility) are practically unaffected by $\dot{\varepsilon}$.

It therefore appears that except for the increased first yielding, no other significant effects of high strain rates on the structure can be expected. This increased yielding strength is usually of no practical consequence when large ductility is built into the structure. Thus, judging from the behaviour of the structure itself, no serious limitations are introduced by testing under equivalent static forces. The same, however, cannot be said of the behaviour of the soil surrounding the foundation. The strain rate effect on soil can be significant and should be considered.

From the above discussion, it is obvious that if the probable seismic response of a building to severe earthquake ground motions is to be studied, the use of equivalent pseudo-static forces, instead of actual dynamic ground motion excitations, are valid only if the foundation deformation of the building has no significant effect on its dynamic response.

(ii) Selection of Test Loading.- Another problem encountered in trying to apply this pseudo-static method of testing is that in real situations the inertia force at each concentrated mass varies with time, depending on the interaction of the real dynamic excitations and the dynamic characteristics of the building. Therefore, simulating the actual inertia forces by simple static forces is very difficult. The only recourse is to simulate what can be considered the critical combination of inertia forces which could develop at a certain time. Rational selection of this critical combination requires integrated experimental and analytical studies because this combination will vary depending upon what we are interested in studying. For example, in tall buildings, the critical combination of inertial forces will depend on the story selected for study. Table 5 illustrates the differences in the loading conditions that were derived using different seismic analysis methods for testing a model of the wall element of a combined frame-wall system (41). The considerable discrepancy in the shear span values points out not only the difficulties in deriving the critical combination of inertia forces, but also the need for careful evaluation of results obtained in experimental investigations when they are interpreted in terms of the actual seismic behaviour of structures.

Furthermore, even if a rational combination of inertia forces can be selected, the problem of how to vary the magnitude of these forces still remains. It is well known that the behaviour of reinforced concrete is very sensitive to the loading path (40,42). Selection of the proper load sequence requires integrating results from both analytical and experimental studies, in other words, analyzing the response of the building using analytical mechanical models obtained from experimental results for loading conditions similar to those encountered in the analytical response. This would require an iterative approach in which at first a simplified mechanical model is assumed to estimate a critical loading sequence. Using this loading sequence in the experiments permits improvements of the mechanical model, which can then be used in a new dynamic-response analysis. This will then lead to the selection of a new loading sequence for the test, and so on.

Japanese researchers have already tested several parts of actual structures up to failure, using repeated reversed pseudo-static lateral forces on buildings up to seven stories (43-45). In these tests important data about how T(t) and ε(t) vary with increased damage in the structure have also been obtained by additional small amplitude free and/or forced vibration tests.

There are too few opportunities to do field tests of actual buildings up to failure, and because of the difficulty of instrumenting and loading these buildings, usually only simple or isolated frames of the structure are tested. Furthermore, because of the sensitivity of soil to strain rate effects, it is possible to study the probable actual seismic behaviour using this type of testing only for the case of a structure on a rigid foundation. Therefore, it is believed that efforts should be devoted to developing pseudo-static facilities that will permit testing of full- or large-scale models of buildings and/or subassemblages of their main structural elements.

4.2.2 Laboratory Test Under Equivalent Pseudo-Static Forces and Additional Small Amplitude Free and Forced Vibration Tests

4.2.2.1 Full-Size Buildings or Large-Scale Models

It does not seem feasible to test the whole soil-structure system of a full-size building in a laboratory. The main purpose of this type of test is not to study the behaviour of one building under one given or selected earthquake ground motion, but rather to obtain information regarding the mechancial behaviour of the structure itself under loading conditions similar to those that might be encountered in actual buildings during severe earthquake ground motions. Therefore, this will require the testing of buildings under generalized types of loadings which would permit predictions of their behaviour under possible severe earthquake ground motions. Once the behaviour of the structure itself is known, the effect of soil-structure foundation can be estimated from analytical parametric studies.

Since 1967, Japanese researchers have been carrying out pseudo-static tests on full-size apartment buildings up to five stories, using the facility illustrated in Fig. 27 (46-48). In most of the tests, repeated reversed lateral forces of a preselected fixed pattern are used. The magnitude of the forces is increased in steps. The advantage of this method is that after each step the building can be subjected to free and/or forced vibration by means of shakers, thereby making it possible at each step to obtain the variation of T and ξ with the amount of damage induced in the building.

The results of these tests have clarified the probable seismic behaviour of very complex structures fabricated from cast-in-place reinforced concrete, precast reinforced concrete, and precast concrete with prestressed construction systems. They have in particular, clarified the interaction between the different elements of these structures in which there were no clearly defined girders and columns so that it is difficult to estimate how much of the slabs and walls contribute to the strength and stiffness of idealized girders, columns, and shear walls. It would have been almost impossible to predict the observed interaction analytically or by means of separate tests of their individual structural components.

Regarding the validity of the results obtained using the above facility, it is important to recognize that the distribution and sequence of the applied forces used might not represent the critical patterns which can be induced in extreme earthquakes. The hysteretic loop for fixed peak deformation depends on the previous history of loading. This is illustrated in Figs. 28(a) and 28(b). If the fixed peak deformations have never been exceeded (Fig. 28(a)), the peak resistance, initial stiffness, and energy dissipation will be larger than those in cases where the fixed peak deformations were exceeded in a previous cycle (Fig. 28(b)). From these observations it appears that the application of repeated reversed loading cycles in which the peak values of the load and/or deformation are increased gradually (the usual method of testing) might not be a "conservative" way of testing. The structure may show considerably less energy dissipation capacity, and even less maximum strength, if it is loaded near or up to its ultimate resistance (deformation) during the first cycle. Therefore, it is obvious that for a better understanding of the possible different seismic behaviour of a building, several full-size or large-scale models of a building must be tested under different types of pseudo-static loading histories.

Large pseudo-static facilities that will permit the testing of full-size buildings or large-scale models should be developed. The need for large-scale rather than small-scale models is due to the fact that the inelastic behaviour of structures -- particularly when reversal of deformations occur -- is very sensitive to the detailing, which is very difficult to simulate at reduced scales. This is particularly true in the case of concrete structures where one of the main reasons for the observed degradation in stiffness and strength under cycles of loading reversals is the bond degradation. The bond characteristics of large deformed bars are dissimilar to those of small bars available in the market.

The pseudo-static facility to be developed should be capable of allowing application of multi-directional deformations or loadings. This could be accomplished with the arrangement illustrated schematically in Fig. 29. This type of facility would permit the application not only of horizontal biaxial deformations, but also of vertical loading by simply attaching auxiliary steel frame elements to the permanent tie down slab and walls.

4.2.2.2 Subassemblages

It would be ideal to test a real building under the actual loading conditions to which it may be subjected during its service life, but such tests are not usually economically feasible. Logically, the next best approach is to try to predict the response of the complete building or its structural system from results obtained in studies carried out on its structural elements. This has been the approach of most investigators. Because of the interaction problem, it is believed that to accurately predict the response of a building under generalized excitations it is necessary to have information regarding the behaviour of certain basic subassemblages of elements. The type of basic sub-assemblage to be studied depends on the structural system used. Several structural systems can be and have been used. For the purpose of this review, however, only two types will be considered: (1) Ductile Moment-Resisting Space Frames, and (2) Shear Walls.

(i) Tests of Planar Subassemblages. - For the case of Ductile Moment-Resisting Space Frame Systems it is necessary to know the behaviour of subassemblages such as those indicated in Fig. 30. Detailed reasons for selecting these subassemblages can be found in Refs. 49 and 50. Testing of these types of subassemblages can be done with facilities similar to those already available. Some of them are illustrated in Figs. 31 and 32 (51,52). To obtain basic information for predicting the in-plane seismic behaviour of shear wall systems, it is necessary to test subassemblages such as those indicated in Fig. 33. The first problem that is encountered in selecting these subassemblages is the simulation of the actual boundary conditions. Solution of this problem usually requires the use of subassemblages with at least two or three shear wall panels. Test setups necessary for studying the in-plane seismic behaviour of these subassemblages can be designed and constructed relatively economically, as illustrated in Fig. 34 (53). A problem that needs careful consideration is the selection of the loading conditions to be applied to the specimens as discussed previously and illustrated in Table 5.

The advantages of carrying out studies on these types of subassemblages are: (1) technically, the mechanical behaviour of what can be considered the basic unit can be studied thoroughly because by carefully planning the instrumentation, it is relatively easy to determine at any time the statics of the whole subassemblage; and (2) economically, it is feasible to carry out extensive and comprehensive studies of most of the different parameters controlling the behaviour of these subassemblages because the cost of the required testing facility, fabrication and testing of specimen, data acquisition and data reduction is low compared to that required for testing buildings dynamically.

Although this method of testing is essential in increasing the knowledge of the role each structural element plays in the overall seismic response of the structure, it has some limitations. The selection of the actual boundary conditions (supports and force applications) and the proper sequence of loading are limiting factors. Bounds of loading histories can, however, be obtained by the proper integration of analytical and experimental studies. The main limitation is that actual buildings suffer three-dimensional displacements and, under this pattern of space deformation, the behaviour of each of the structural elements, as well as their interaction, cannot be predicted from tests in just one plane. Thus, the tests of planar subassemblages should be supplemented by the following types of experimental studies.

(ii) Static and Dynamic Tests of Space Sub-Assemblages - One- to three-story space subassemblages should be tested by subjecting them to forces in one vertical and two horizontal directions. This can be accomplished by developing three-dimensional pseudo-static testing facilities, such as the one illustrated in Fig. 29. The instrumentation of the test specimens should be designed to permit the determination of the internal forces in all the structural members at any loading stage. It will be possible to vary the vertical forces according to the variations which might be expected due to the effects of overturning moments induced by the upper part of the building to which the sub-assemblage belongs. It will be difficult, however, to study possible effects of the potential vertical component of the ground acceleration with this kind of facility. The main difficulty arises from the fact that the response to the vertical component of the ground acceleration usually varies with a higher frequency than that corresponding to the horizontal motion. Therefore, some significant strain rate effects may be induced. Because recent analytical studies (4,54) indicate that the vertical motion can be particularly significant in increasing ductility demands in girders and columns of the upper stories of tall buildings, and inspection of damages during recent severe earthquakes appears to confirm this, there is a need for studying behaviour of structures under simultaneous application of horizontal and vertical components of the ground motion. Thus, it is suggested that the effects of the vertical component of the ground motion be investigated by carrying out tests on shaking tables which can simulate the vertical, as well as at least one horizontal, component of the ground motion. The main purpose of these tests would be to study the bounds of possible effects rather than the response of an actual building to a real earthquake motion. Testing could be carried out on very simple 1 or 2 bay tower type of subassemblages and would not require very large shaking tables.

The most feasible way of rapidly gaining knowledge regarding the effects of simultaneous action of the three components of the ground motion is by integrating the results of the types of experiments described above with three-dimensional analyses of structures. Although computer programs for carrying out nonlinear three-dimensional analyses are now becoming available, caution should be used in interpreting the results obtained from such programs, because not only are they based on highly idealized three-dimensional hysteretic behavioural models, but also because the results can be very sensitive to variations in the characteristics of each of the three components of ground motions that must be fed into the computer. Unfortunately, little guidance is available at present regarding these two problems.

5. RELIABILITY ANALYSIS

Because of the uncertainties that have been pointed out in previous discussions regarding the characteristics of future major earthquake ground shaking, as well as the actual mechanical behaviour of the soil-structure system, the nature of an aseismic design is nondeterministic. Therefore, it is necessary to subject the designed structure to a series of analyses to check its reliability under the possible bounds of the expected excitations and of the parameters controlling its behaviour at service and at ultimate limit states [9]. The following discussion will be limited to the problems that arise in checking the reliability of the design at the ultimate limit states.

Because the nonlinear dynamic response of structures is very sensitive to variations in the characteristics of ground motions [9,54], the reliability of a design against a suite of ground motion time-histories should be checked. These time-histories should be selected in such a way that it will test the inelastic response of the structure throughout the probable range of potentially critical periods in which it can respond due to the degradation of its stiffness.

Quantitative description of the actual hysteretic behaviour of a member or its critical regions is complex. Most of the data available are from tests of members under moment, axial and shear forces acting in one plane. Even for this simple case of planar behaviour, modelling of the real hysteretic loops is still too complex for incorporation into practical computer programs for the analysis of whole structures. Thus, it is desirable to describe its main characteristics by a few numerical indices. If this behaviour can be idealized as being elasto-perfectly plastic, it can be precisely described by the yield strength and time-history of a ductility factor, defined as the system deformation divided by its yield deformation. Unfortunately, the hysteretic behaviour of real systems usually differs significantly from this simple idealization (Fig. 35). Thus, although ductility factors describe the maximum deformations, they generally fail to quantify the energy dissipation capacity.

5.1 Use of Ductility Factors

In structural analysis two types of ductility factors are used: (1) displacement ductility ratios determined to estimate overall response, and (2) curvature or rotation ductility factors computed to study the behaviour of individual critical regions. The advantages and disadvantages of using these factors for analysing the reliability of an aseismic design are discussed in Ref. 55. A summary of the conclusions reached in this study follows:

5.1.1 Ductility Factors for Overall Response. - Recently suggested preliminary design methods use overall lateral displacement ductility factors to determine design forces [9]. Damage control is a main consideration in selecting design ductility factors, since expected damages increase with the value of these factors. Ductility factors based on horizontal floor displacements or story drifts may not adequately reflect the true damage potential, however, since they include the substantial horizontal displacement component resulting from axial column deformations which is not a usual source of damage (Fig. 36(a)). A better damage index is the tangential story drift index, R (Fig. 36(b)). Since yield displacements are not well defined for realistic systems, displacement ductilities are only approximate.

5.1.2 Ductility Factors for Critical Regions. - To design and detail the critical regions of a structure, it would be ideal to study their entire hysteretic behaviour. Since this is generally impractical, it is desirable to have a measure of the maximum inelastic deformations in each direction, and the magnitude and number of severe inelastic reversals. The plastic rotations and curvatures that are developed may be used for this purpose. It is difficult to determine meaningful yield rotations, however, and computations of the plastic rotation capacity are complex, requiring knowledge of the critical region's length, moment variation and moment-curvature relationships. On the other hand, yield and ultimate curvatures depend only on the section properties, and cyclic curvature ductility factors have been suggested to account for the effect of reversed plastification [56]. Thus, it would appear advantageous to use curvatures as comparative indices of inelastic behaviour.

5.2 Analytical Prediction of Curvature Ductility Factors

In nonlinear structural analyses, inelastic deformations are usually assumed to occur at concentrated plastic hinges (lumped plasticity models). One of the most extensively used analytical techniques to simulate the flexural behaviour of members with bilinear hysteretic moment-curvature ($M-\phi$) characteristics is the two-component model shown in Fig. 37 [56].

5.2.1 Two-Component Model. - When flexural members with bilinear $M-\phi$ relationships are subjected to moment gradients, plastic deformations will be concentrated at discrete points only if the rate of strain-hardening equals zero. Thus, idealizations, such as the two-component model, cannot exactly represent the behaviour of flexural members with bilinear sectional stiffness characteristics [57].

The effects of assuming lumped plasticity, rather than the more realistic spread plasticity, has been quantitatively evaluated in Ref. 56 for the beam shown in Fig. 33, by comparing results obtained using a two-component model with the exact solution based on bilinear sectional stiffnesses. These results reveal that:

(1) the two-component model idealization may substantially underestimate the stiffness of the member. This is illustrated in Fig. 39 where the values of the tangent stiffness, K_T (normalized with respect to the elastic stiffness, K_{EL}), for the realistic model and the two-component model are plotted

versus the curvature ductility factor, μ_ϕ^R, for the realistic model of the beam, illustrated in Fig. 38.

(2) curvature ductility estimates made on the basis of two-component idealizations may substantially underestimate actual ductility requirements, particularly at low values of ductility and strain-hardening. This is clearly shown in Fig. 40, which plots the values of the ratio between the curvature ductility factors obtained from the two-component model, μ_ϕ^M, and the stiffness, p (see Fig. 37). The approximate relationship plotted in this figure has been obtained assuming that the length of the plastic region (L_p in Fig. 38) is very small compared to total length of beam, L.

5.2.2 Reliability of Two-Component Model. - In view of these findings, it is necessary to carefully evaluate and interpret results obtained assuming lumped plasticity. Ductility factors, as usually computed at present, are useful as design guidelines but they must be first carefully defined and interpreted. There is an urgent need to investigate the dynamic response of structures, accounting for spread plastification.

6. CONCLUSIONS

From the above review and previous discussions, it can be concluded that in order to improve the earthquake resistant design of building structures, it would be necessary:

1. To continue installing strong motion seismographs. Only the continued accumulation of statistical evidence can lead to improved estimates of the extreme ground shaking at the foundation of buildings and thereby allow proper design earthquakes to be established. Specification of the severity of ground shaking by a site peak ground acceleration alone is inadequate for establishing rational design earthquakes.
2. To develop more reliable inelastic design response spectra. Pending more statistical evidence, there is an urgent need to establish the largest pulse (largest incremental velocity and associated average acceleration) that can be transmitted by each of the different layers of soil in which a structure could be supported, in order to improve, in the shortest possible time, present methods of establishing design earthquakes and inelastic design response spectra.
3. To design reliable devices (isolation mechanisms) for controlling the ground motion input to a building. This will require testing of these devices in earthquake simulator facilities (shaking tables). The larger the capacity of these facilities, the more reliable the results.
4. To investigate mechanical behaviour or structural materials under dynamic earthquake-like conditions.
5. To improve quality control of materials and to formulate more stringent code specifications regarding design material characteristics.
6. To develop earthquake simulator facilties large enough to test full-size non-engineered buildings. These facilities should be supplemented with the development of large pseudo-static (or pseudo-dynamic) facilties.
7. To study the overall earthquake resistant characteristics of new structural systems by testing medium-scale models using medium-scale shaking tables.
8. To develop and test damping devices which, when installed between structural components of a building, can control the buildings response. The testing of these devices should be carried out on shaking tables, the larger the capacity, the more reliable the results.
9. To improve knowledge of the mechanical behaviour of the whole soil-structure system under extreme earthquake environments. This would require the following:

(a) Thorough instrumentation of actual buildings and their surroundings (soil) in all seismic areas.
(b) Testing of building-soil systems by subjecting them to extreme earthquake-like environments by means of underground explosions. These types of tests are essential in order to understand the actual behaviour of foundations. Although considerable information regarding this behaviour can be gained from pseudo-static tests, dynamic tests are needed due to the high sensitivity of the mechanical characteristics of the soil to strain rate and the existence of radiation damping.
(c) Development of pseudo-static (or pseudo-dynamic) facilities that would permit testing under pseudo-static forces, three-dimensional subassemblages, and models of whole buildings in full- or large-scale.
(d) Continuation of testing planar subassemblages of different structural systems under pseudo-static forces which can induce effects similar to those expected from severe earthquake shaking.
(e) Testing of full- or large-scale models of simple space, structural subassemblages in a medium-scale earthquake ground simulator which could simultaneously simulate the vertical component and at least one of the two horizontal components of possible earthquake ground motions.
(f) To carry out post-earthquake analyses of damages in order to identify reasons for the observed damages and thereby to improve knowledge of the actual mechanical behaviour, and to assess the reliability of various analytical models and techniques available for predicting structural response.

10. To develop suitable (realistic) mathematical models of the nonlinear behaviour of different structural systems for carrying out reliability analyses. This requires experiments using realistic physical models.
11. To integrate analytical and experimental research in order to determine the ductility demands and the ductility available for different structural systems that are used, or can be used, in aseismic design. The ultimate objective of this is to establish what controls the design (the maximum tolerable deformation due to the expected damage, or the available ductility) and what can be considered an acceptable ductility ratio for the design of each of these different structural systems.
12. To improve coordination and integration of national and international research programs in the field of earthquake engineering.
13. To translate the research results into information useful to design engineers and code officials. Because of limited funds, a list of research priorities should be

prepared. From the list of needs presented above, the author believes the following research priority list to be desirable: 12, 13, 11, 1, 2, 9, 10, 6, 5, 4, 3, 8, 7.

REFERENCES

1. Pinkham, C., "Procedures and Criteria for Earthquake Resistant Design", Building Practices for Disaster Mitigation, National Bureau of Standards, Building Science Series 46, February 1973, pp. 188-208.
2. Bertero, V. V., "Experimental Studies Concerning Reinforced, Prestressed and Partially Prestressed Concrete Structures and their Elements", Introductory Report for Theme IV of the IABSE Symposium on Resistance and Ultimate Deformability of Structures Acted on by Repeated Forces, Lisbon, 1973.
3. Leeds, D. J., "The Design Earthquake", Geology Seismicity and Environmental Impact, Special Publication, Association of Engineering Geologists, University Publishers, Los Angeles, California, October 1973, pp. 333-347.
4. Rosenblueth, E., "The Six Components of Earthquakes", Proceedings of the Australian Conference on Planning and Design of Tall Buildings, Sydney, Australia, August 14-17, 1973, pp. 63-81.
5. "An Evaluation of a Response Spectrum Approach to Seismic Design of Buildings", Applied Technology Council, San Francisco, California, September 1974, 173 pp.
6. Newmark, N., Blume, J., and Kapur, K., "Seismic Design Spectra for Nuclear Power Plants", Journal of the Power Division, ASCE, Vol. 99, No. PO2, Proc. Paper 10142, November 1973, pp. 287-303.
7. Wesson, R. L., et al., "Expectable Earthquakes and their Ground Motions in the Van Norman Reservoir Area", U.S. Geological Survey, Circ. 691-A,B, pp. B_1-B_9.
8. Page, R. A., et al., "Ground Motion Values for Use in the Seismic Design of the Trans-Alaska Pipeline System", U.S. Geological Survey, Circ. 672, 1972, 23 pp.
9. Bertero, V. V. and Kamil, H., "Nonlinear Seismic Design of Multistory Frames", paper to be published in the Canadian Journal of Civil Engineering, National Research Council of Canada, Toronto, December, 1975.
10. Newmark, N. M. and Hall, W. J., "Procedures and Criteria for Earthquake Resistant Design" Building Practises for Disaster Mitigation, National Bureau of Standards, Building Science Series 45, February 1973, pp. 209-237.
11. Bertero, V. V., et al., "Design Implication of Damage Observed in the Olive View Medical Center Building", Proceedings of the Fifth World Conference on Earthquake Engineering, Vol. 1, Rome, June 1973, p.p. 51-59.
12. Reimer, R. B., R. W. Clough, and J. M. Raphael, "Evaluation of the Pacoima Dam Accelerogram", Proceedings of the Fifth World Conference on Earthquake Engineering, Vol. 2, Rome, June 1973, pp. 2328-2337.
13. Bertero, V. V. and Mahin, S., "Some Observations Regarding Establishment of Design Earthquakes", Earthquake Engineering Research Center, University of California, Berkeley. Report in preparation.
14. Ambraseys, N. N., "Maximum Intensity of Ground Movements Caused by Faulting", Procéedings of Fourth World Conference on Earthquake Engineering, Vol. 1, Santiago, Chile, 196, pp. 154-171.
15. Brune, J. N., "Tectonic Stress and the Spectra of Seismic Shear Waves from Earthquakes", Journal Geophysical Research, Vol. 75, 1970, pp. 4997-5009.
16. Matsushita, K., and Izumi, M., "Some Mechanisms to Decrease Earthquake Effect to Buildings", Proceedings of the Fourth World Conference on Earthquake Engineering, Vol. 1, Santiago, Chile, 1969.
17. Wirsching, P. H. and Yao, J. T. P., "A Statistical Study of Some Design Concepts in Earthquake Engineering", University of New Mexico, Tech. Report CE-21, 1970.
18. Matsushita, K. and Izumi, M., "Application of Input Controlling Mechanism to Structural Design of a Tall Building", Proceedings of the Fifth World Conference on Earthquake Engineering, Vol. 2, Rome, June 1973, pp. 2948-2955.
19. Martel, R. R., "The Effects of Earthquakes on Buildings with a Flexible First Story", Bulletin of Seismic Society of America, Vol. 19, No. 3, 1929.
20. Fintel, M. and Kahn, R. F., "Shock Absorbing Soft Story Concept for Multistory Earthquake Structures", ACI Journal, No. 66-29, 1970, pp. 318-390.
21. Bertero, V. V., "Studies of Olive View Building Damage", paper presented at the U.S.-Japan Seminar on Earthquake Engineering with Special Emphasis on Reinforced Concrete Structures, Berkeley, California, Sept. 4-8, 1973.
22. Chopra, A. K., et al., "Earthquake Resistance of Buildings with a Soft First Story", Earthquake Engineering and Structural Dynamics, Vol. 1, 1973, pp. 347-383.
23. Norris, C. H., et al., "Structural Design for Dynamic Loads", McGraw-Hill, New York, 1959, Chapters 1 and 2.
24. Air Force Design Manual, "Principles and Practices for Design of Hardened Structures", Technical Documentary Report No. AFSWC-TDR-62-138, December 1962, Chapter 6.
25. Newmark, N. and Rosenblueth, E., "Fundamentals of Earthquake Engineering", Prentice Hall, Inc., 1971.
26. Bresler, B. and Bertero, V. V., "Olive View Medical Center Material Studies - Phase I", Earthquake Engineering Research Center, Report No. EERC 73-19, University of California, Berkeley, December 1973, pp. 92.
27. Bertero, V. V. and Collins, R. G., "Investigation of the Failures of the Olive View Stairtowers During the San Fernando Earthquake and their Implications on Seismic Design", Earthquake Engineering Research Center, Report No. EERC 73-26, University of California, Berkeley, December 1973, pp. 282.
28. Krawinkler, H., Bertero, V. V. and Popov, E. P., "The Beam-Column Joint

- An Important Element in the Seismic Response of Moment Resisting Steel Frames", paper submitted for publication in the Proceedings of the Fifth European Conference on Earthquake Engineering, Ankara, Turkey, September 1975.

29. Bresler, B. and Bertero, V. V., "Influence of High Strain Rate and Cyclic Loading on Behaviour of Unconfined and Confined Concrete in Compression", Proceedings of the Second Canadian Conference on Earthquake Engineering, McMaster University, Hamilton, Ontario, June 1975, pp. 64.
30. "Response of Multistory Concrete Structures to Lateral Forces", ACI Publication SP-36, 1973.
31. Kelly, J. M., Skinner, R. I. and Heine, A. J., "Mechanisms of Energy Absorption in Special Devices for use in Earthquake Resistant Structures", Bulletin of N.Z. Society for Earthquake Engineering, Vol. 5, No. 3, September 1972, pp. 63-88.
32. Bertero, V. V., Mahin, S. A. and Hollings, J., "Response of a Reinforced Concrete Shear Wall Structure During the 1972 Managua Earthquake", Bulletin of N.Z. National Society for Earthquake Engineering, Vol. 7, No. 3, September, 1974.
33. "Managua, Nicaragua Earthquake of December 23, 1972", Engineering Research Institute, Conference Proceedings, Vol. I and II, San Francisco, November 29 and 30, 1973.
34. Kustu, O. and Bouwkamp, J. G., "Behaviour of Reinforced Concrete Deep Beams-Column Subassemblages Under Cyclic Loading", Earthquake Engineering Research Center, Report No. EERC 75-11, University of California, Berkeley, 1975.
35. Osawa, Y., Tanaka, T., Murakami, M. and Kitagawa, Y., "Earthquake Measurements in and Around a Reinforced Concrete Building", Proceedings of the Fourth World Conference on Earthquake Engineering, Volume 1, Santiago, Chile, 1969, B-1, pp. 1-16.
36. Kitagawa, Y., "A Study on the Soil-Building Interaction System by Simultaneous Observation of Earthquake Motions", Building Research Institute, Tokyo, Japan, April 1972.
37. Iemura, H. and Jennings, P. C., "Hysteretic Response of a Nine-Story Reinforced Concrete Building during the San Fernando Earthquake", Report No. EERL 73-07, California Institute of Technology, October 1973, 55 pp.
38. Bernreuter, D. L., and Tokarz, F. J., "Providing an Earthquake Like Environment for Testing Full-Scale Structures by Using the Ground Motion from Underground Nuclear Tests", Proceedings of the Fifth World Conference on Earthquake Engineering, Vol. 1, Rome, June 1973, pp. 230-233.
39. Penzien, J., et al., "Feasibility Study of Large Scale Earthquake Simulator Facility", Earthquake Engineering Research Center, Report No. EERC 67-1, University of California, Berkeley, 1967.
40. Bertero, V. V., et al., "Rate of Loading Effects on Uncracked and Repaired Reinforced Concrete Members", Proceedings of the Fifth World Conference on Earthquake Engineering, Vol. 2, Rome, 1973, pp. 1461-1470.
41. Wang, T. Y., Popov, E. P. and Bertero V. V., "Hysteretic Behaviour of Reinforced Concrete Framed Walls", Earthquake Engineering Research Center, University of California, Berkeley, report in preparation.
42. Bertero, V. V., Bresler, B. and Liao, H., "Hysteretic Behaviour of Reinforced Concrete Members Subjected to Cyclic Flexural Moment", Earthquake Engineering Research Center, Report No. EERC 69-12, University of California, Berkeley, December 1969, 115 pp.
43. Nakagawa, H. and Hirosawa, M., "Experimental Results of an Actual Six Storied Reinforced Concrete Frame", Building Research Institute, Tokyo, Japan, 1965, 42 pp.
44. Funahashi, I., et al., "Vibration Tests and Test to Failure of a Seven Storied Building Survived a Severe Earthquake", Proceedings of the Fourth World Conference on Earthquake Engineering", Vol. 1, Santiago, Chile, 1969, B-1 pp. 26-43.
45. Aoyama, H., et al., "A Study of Damage to the Hachinohe Technical College Due to 1968 Tokachi-Oki Earthquake (Part I)", Proceedings of the U.S.-Japan Seminar on Earthquake Engineering with Emphasis on the Safety of School Buildings, Sendai, Japan, September 1970, pp. 199-213.
46. Ohsaki, Y., et al., "Experimental Study on Five-Story Full-Size Apartment House of Reinforced Concrete Walled Frames", Proceedings of the U.S.-Japan Seminar on Earthquake Engineering with Emphasis on the Safety of School Buildings, Sendai, Japan, September 1970, pp. 240-266.
47. Shinagawa, T., and Hirosaw, M., "Lateral Load Test on Five-Storied Full-Size Building of Precast Reinforced Concrete Wall Construction", Seminar Under the Japan-U.S. Co-operative Science Program Construction and Behaviour of Precast Concrete Structures, Reports by Japanese Participants Part I, Seattle, Oregon, August 23-27, 1971, pp. 91-102.
48. Nakano, K., et al., "Structural Behaviour of Multi-Storied Precast Reinforced Concrete Construction Assembled with Post-Tensioning Method", Seminar Under the Japan-U.S. Co-operative Science Program Construction and Behaviour of Precast Concrete Structures, Reports by Japanese Participants, Part I, Seattle, Oregon, August 23-27, 1971, pp. 91-102.
49. Bertero, V. V., "Research Needs in Limit Design of Reinforced Concrete Structures", Earthquake Engineering Research Center, Report No. EERC 71-4 University of California, Berkeley, June 1971, 41 pp.
50. Bertero, V. V., "Inelastic Behaviour of Beam-to-Column Subassemblages Under Repeated Loading", Earthquake Engineering Research Center, Report No. EERC 68-2, University of California, Berkeley, April 1968.
51. Bertero, V. V. and Popov, E., "Testing Facility for Frame Subassemblages", Experimental Mechanics, Journal of the Society of Experimental Stress Analysis, Vol. XXVIII, August 1971.
52. Bouwkamp, J. G. and Kustu, O., "Experimental Study of Spandrel Wall Assemblies",

Proceedings of the Fifth World Conference on Earthquake Engineering, Vol. 1, Rome, June 1973, pp. 782-290.
53. Bertero, V. V., Popov, E. P. and Endo, T., "Testing Facility for Subassemblages of Structural Wall Systems", Earthquake Engineering Research Center, University of California, Berkeley. Report in preparation.
54. Anderson, J. C. and Bertero, V. V., "Effects of Gravity Loads and Vertical Ground Acceleration on the Seismic Response of Multistory Frames", Proceedings of the Fifth World Conference on Earthquake Engineering, Vol. II, Rome, June 1973, pp. 2914-2919.
55. Mahin, S. and Bertero, V. V., "Use of Ductility Factors in Aseismic Design", submitted for publication in Proceedings of the Fifth European Conference on Earthquake Engineering, Ankara, Turkey, September 1975.
56. Mahin, S. A. and Bertero, V. V., "An Evaluation of Some Methods for Predicting Seismic Behaviour of Reinforced Concrete Buildings", Earthquake Engineering Research Center Report No. EERC 75-5, University of California, Berkeley, February 1975.
57. Powell, G. H., "Notes on Inelastic Dynamic Analysis of Tall Buildings", Short Course on Earthquake-Resistant Design of Engineering Structures, Continuing Education in Engineering, University of California Extension, Berkeley, June, 1972.

TABLE 1. GENERAL ASPECTS INVOLVED IN DESIGN AND CONSTRUCTION OF AN EARTHQUAKE-RESISTANT BUILDING

DESIGN	CONSTRUCTION
1. ESTABLISHMENT OF DESIGN EARTHQUAKES 2. SELECTION OF STRUCTURAL MATERIAL AND STRUCTURAL SYSTEM	6. QUALITY CONTROL OF MATERIALS
3. PREDICTION OF MECHANICAL BEHAVIOR OF STRUCTURE: (1) MODELING OF STRUCTURE, (2) DESIGN METHOD	7. WORKMANSHIP
4. PROPORTIONING AND DETAILING OF MEMBERS, CONNECTIONS AND SUPPORTS	8. FIELD INSPECTION
5. RELIABILITY ANALYSIS	

TABLE 2 - SUMMARY OF STATISTICAL EVALUATION OF REINFORCING STEEL MILL TEST DATA

	Average Value $\bar{x}$	Standard Deviation σ	ASTM Specified Minimum x_{min}	Excess[(1)] Ratio t_1	% Below Specified Minimum (estimated)
A-15 Grade					
Yield Strength, ksi	52.7	3.2	40	3.97	< 0.5
Tensile Strength, ksi	80.1	4.5	60	4.5	0.75
% Elongation	19.4	3.3	Varies	1.8 to 3.8	2.5 to < 0.5
A-432 Grade					
Yield Strength, ksi	71.1	6.7	60	1.7	4
Tensile Strength, ksi	112.5	8.8	90	2.6	0.5
% Elongation	12.4	2.8	Varies	0.9 to 6.4	19 to < 0.75

(1) $t_1 = (\bar{x} - x_{min})/\sigma$

TABLE 3 - COMPARISON OF OBSERVED AND CALCULATED VALUES OF MODULUS OF ELASTICITY FOR CONCRETE IN COMPRESSION

Specimen Group	Specified Strength f'_c psi	Observed - Average Values			Calculated Values			
		Strength $f_{c\ ave}$ psi	Unit Weight w pcf	E_{obs} ksi	$E_c^{(1)}$ ksi	$E_{obs}/E_c^{(1)}$	$E_c^{(2)}$ ksi	$E_{obs}/E_c^{(2)}$
MTF I	5000	6700	144	3420	4030	0.85	4660	0.73
MTF III(3)	5000	9340	148	3880	4200	0.92	5740	0.68
MTF III(4)	3000	5690	147	3110	4160	0.75	4440	0.70
PDC	3000	4830	113	2040	2170	0.94	2750	0.74
ST	5000	7410	146	3410	4140	0.82	5040	0.67
AC	3000	4280	145	2720	3110	0.88	3710	0.73
Average						0.86		0.71

(1) E_c calculated on the basis of specified strength

(2) E_c calculated on the basis of observed strength

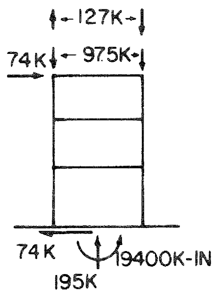
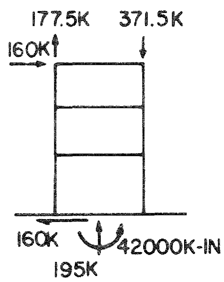
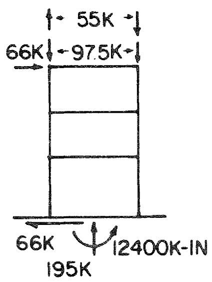
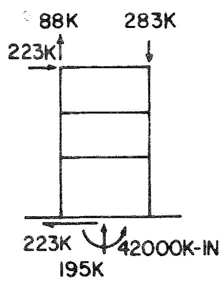
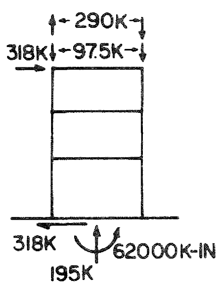
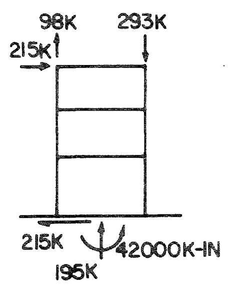
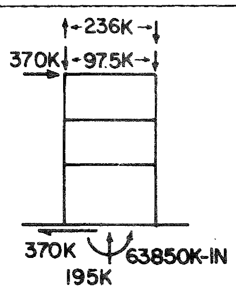
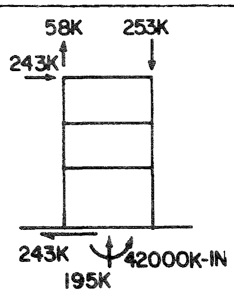
(3) From columns in first story; see Table 6

(4) From beam at second floor level; see Table 6

TABLE 4 EFFECT OF CONFINEMENT ON COMPRESSIVE STRENGTH AND DEFORMATION OF CONCRETE

Type of Concrete	Confinement Stress Ratio (f_r/f'_c)	Maximum Compression		Ultimate Compression	
		Strain Ratio $(\varepsilon_0^*/\varepsilon_0)$	Confinement Effectiveness k_0	Strain Ratio $(\varepsilon_u^*/\varepsilon_0)$	Confinement Effectiveness k_u
Normal E-5	0.13	2.8	7.0	11.5	0
	0.32	7.8	5.0	11.5	3.1
Lightweight R-5	0.13	1.9	4.4	8.7	-0.5
	0.32	4.0	2.0	6.7	2.0
B-5	0.13	1.35	3.9	10.6	0
	0.32	1.85	1.0	8.6	0.9
R-3	0.11	1.8	2.7	8.9	-1.0
	0.24	5.9	2.5	8.9	2.0
B-3	0.11	1.7	1.35	11.6	0
	0.24	8.0	2.1	9.0	2.1

TABLE 5 COMPARISON OF LOADING CONDITIONS FOR MODEL DERIVED FROM ANALYZING PROTOTYPE STRUCTURES USING DIFFERENT SEISMIC ANALYSIS METHODS

DESIGN CRITERIA	METHODS OF DETERMINING SEISMIC FORCES		SIMPLIFIED LOADING CONDITIONS FOR WALL SUBASSEMBLAGE MODEL: COMPUTED FORCES	SIMPLIFIED LOADING CONDITIONS FOR WALL SUBASSEMBLAGE MODEL: ULTIMATE FORCES BASED ON ESTIMATED FLEXURE STRENGTH OF 42,000 K-IN.	SHEAR SPAN a(IN) $(\frac{a}{d})$
WALLS ALONE RESIST TOTAL SEISMIC LATERAL FORCES	UBC				263 (2.8)
FRAMES AND WALLS RESIST TOTAL SEISMIC LATERAL FORCES	UBC				139 (2.0)
	LINEAR ELASTIC RESPONSE SPECTRUM $\ddot{U}_g = 0.33g$ and $\xi = 5\%$	FIRST MODE			195 (2.1)
		FIRST THREE MODES			173 (1.9)

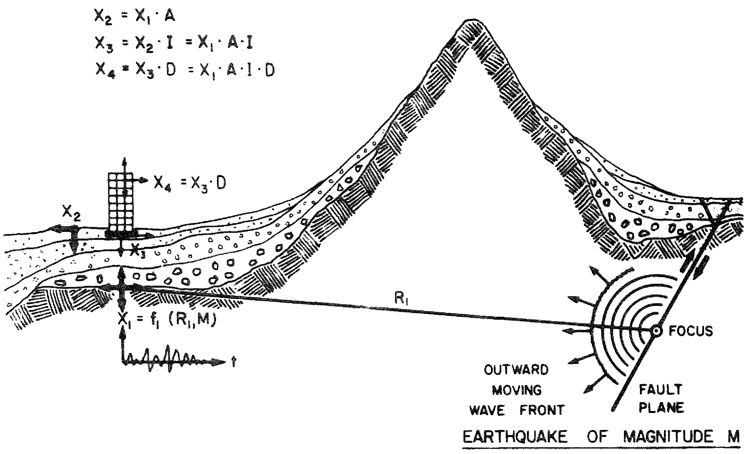


FIGURE 1: FACTORS INVOLVED IN PREDICTING SEISMIC RESPONSE

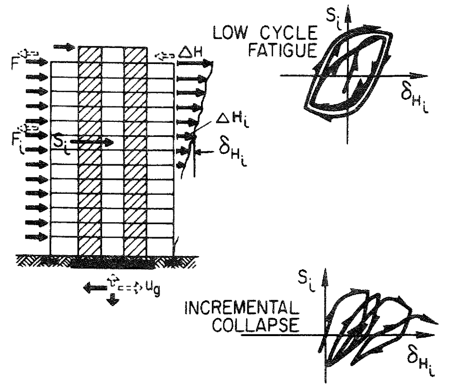


FIGURE 2: STORY SHEAR DISPLACEMENT RELATIONSHIP

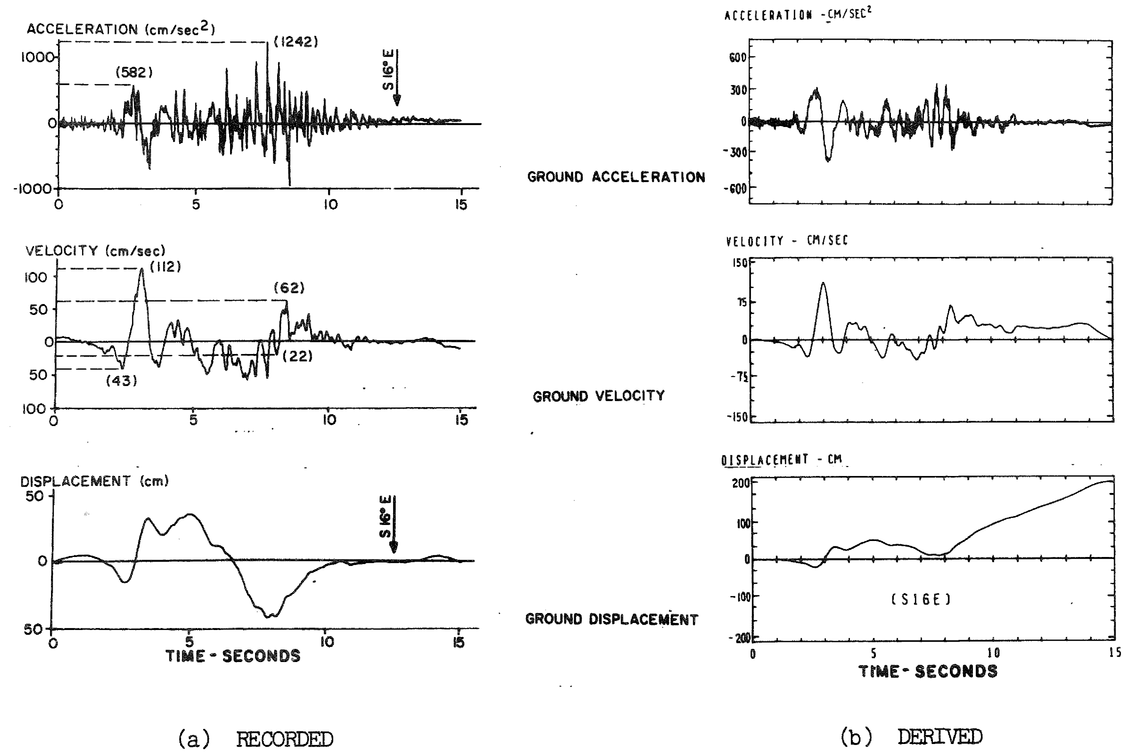


FIGURE 3: SAN FERNANDO EARTHQUAKE GROUND MOTIONS, PACOIMA DAM

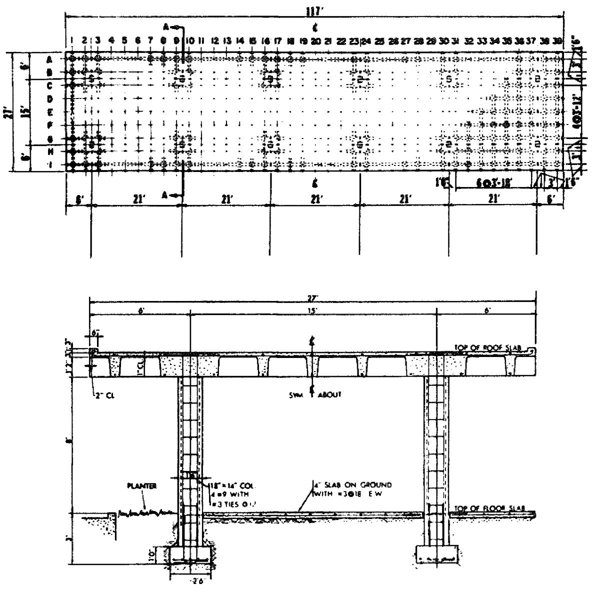


FIGURE 4: ROOF FRAMING PLAN AND SECTION A-A OF AMBULANCE CANOPY

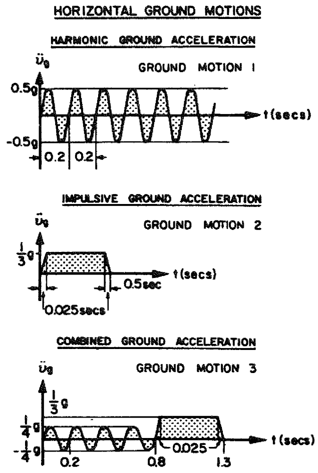


FIGURE 5: GROUND MOTIONS

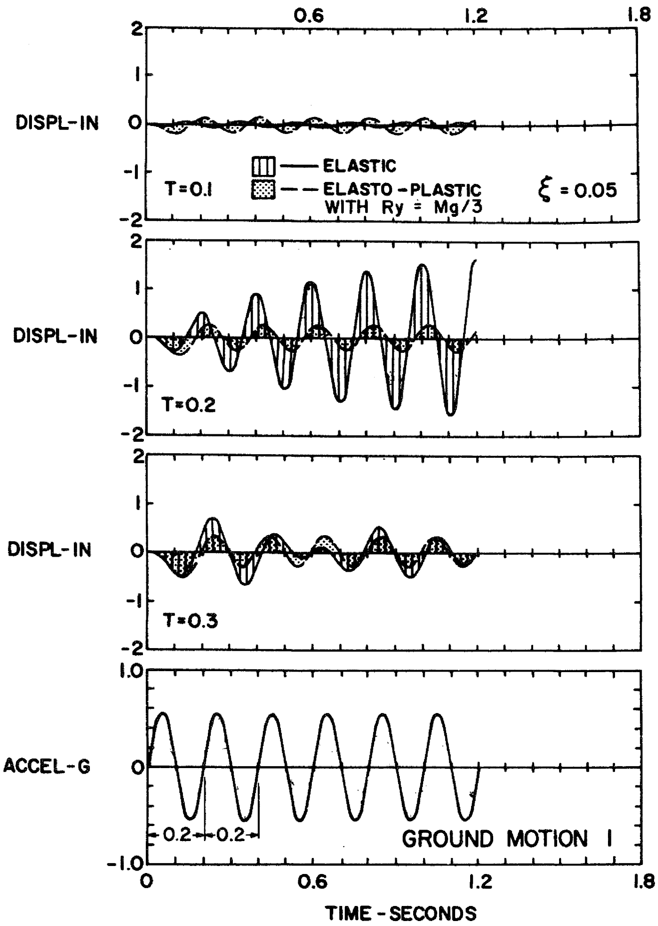


FIGURE 6: RELATIVE DISPLACEMENT TIME-HISTORIES - GROUND MOTION 1

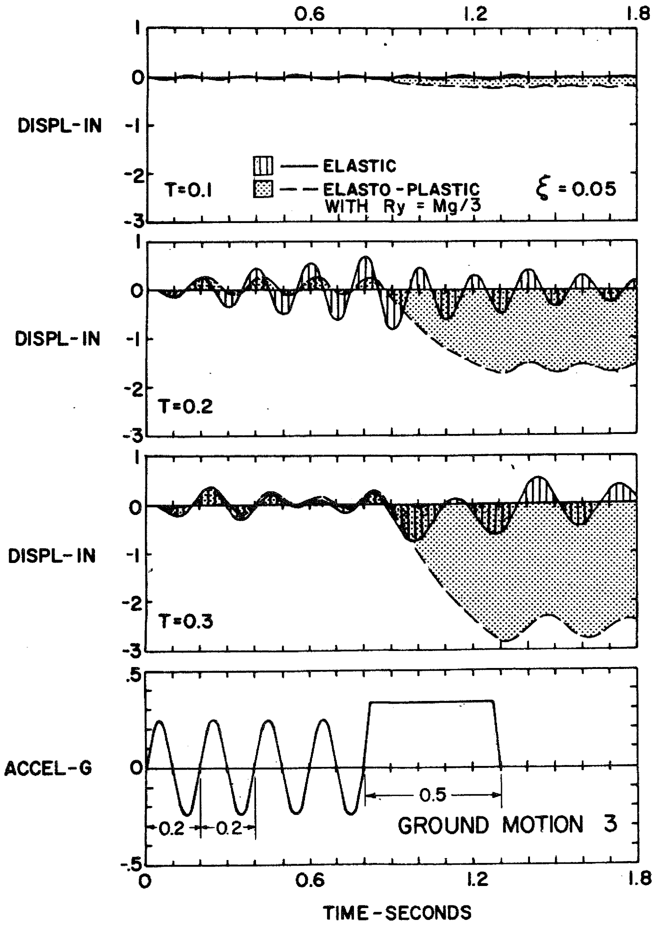


FIGURE 7: RELATIVE DISPLACEMENT TIME-HISTORIES - GROUND MOTION 3

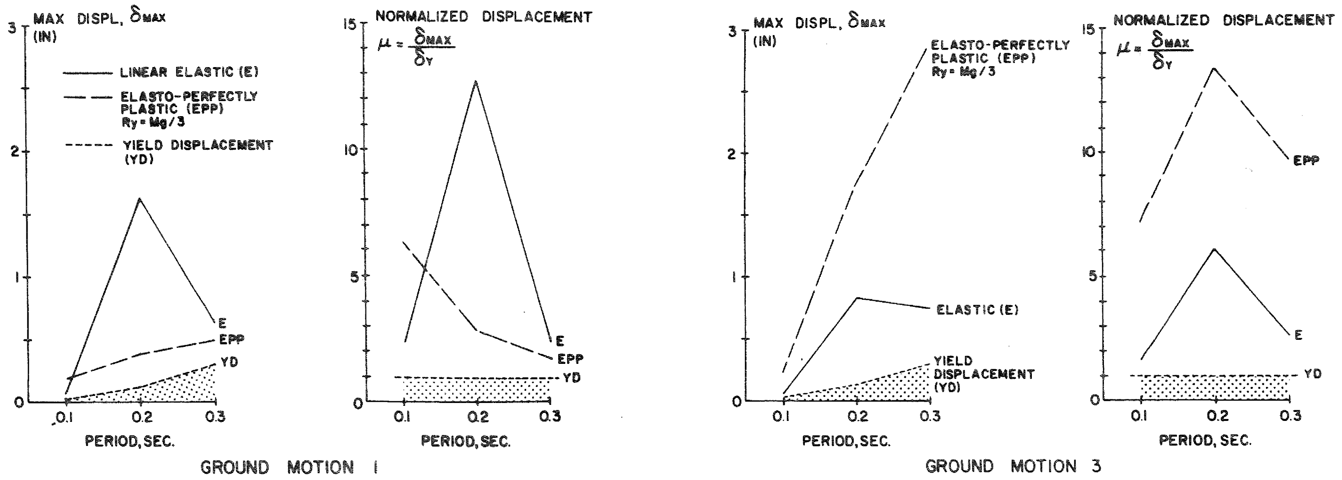


FIGURE 8: MAXIMUM DISPLACEMENT AND DUCTILITY VS. PERIOD

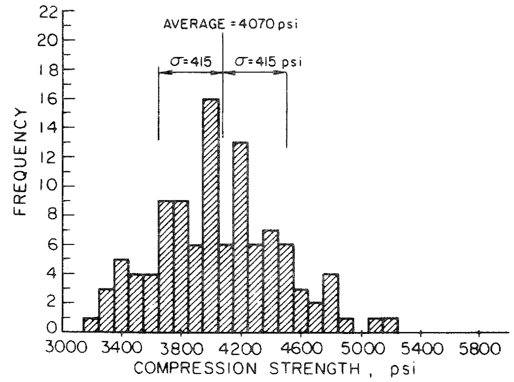


FIGURE 9: HISTOGRAM OF CONCRETE COMPRESSIVE STRENGTH-MAIN TREATMENT FACILITY BUILDING - FIELD CONTROL TESTS - 3000 PSI CONCRETE

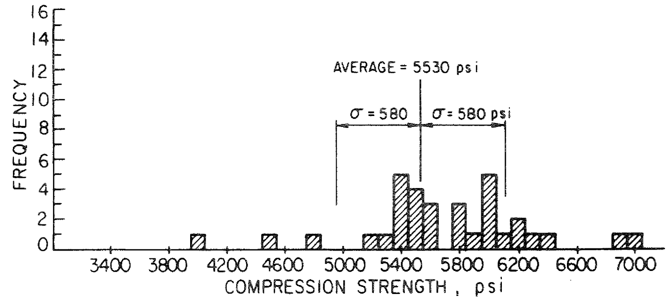


FIGURE 10: HISTOGRAM OF CONCRETE COMPRESSIVE STRENGTH - MAIN TREATMENT FACILITY BUILDING - FIELD CONTROL TESTS - 5000 PSI CONCRETE

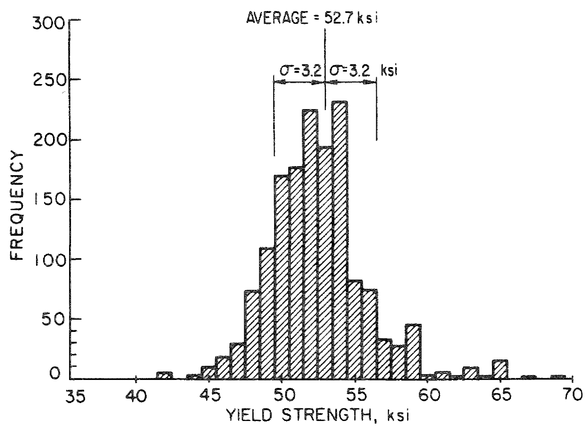


FIGURE 11: HISTOGRAM OF YIELD STRENGTH OF REINFORCING STEEL A-15 GRADE - MILL TESTS

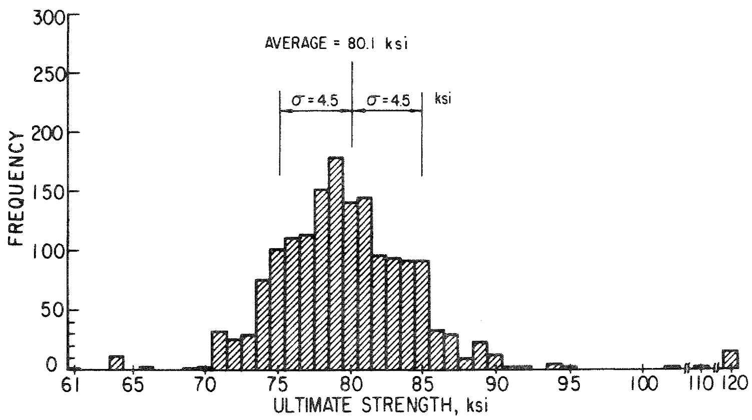


FIGURE 12: HISTOGRAM OF TENSILE STRENGTH OF REINFORCING STEEL A-15 GRADE - MILL TESTS

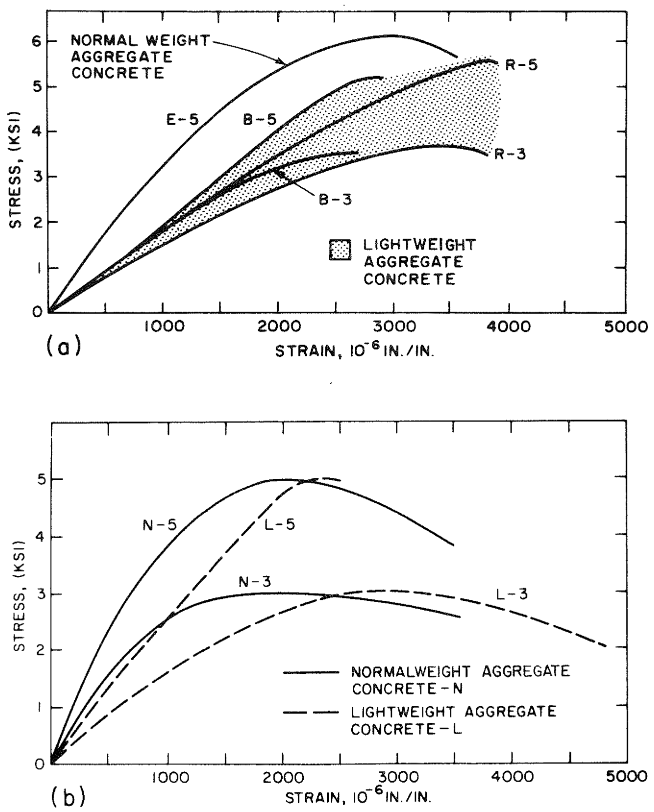


FIGURE 13: STRESS-STRAIN DIAGRAMS FOR UNCONFINED CONCRETE - MONOTONIC COMPRESSION

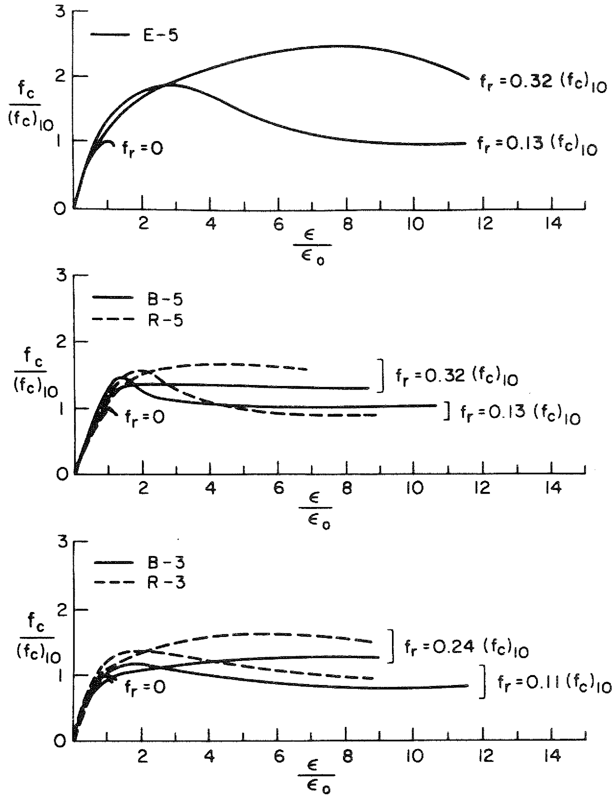


FIGURE 14: EFFECT OF CONFINEMENT PRESSURE ON COMPRESSIVE STRENGTH AND DUCTILITY OF CONFINED CONCRETE - MONOTONIC LOADING

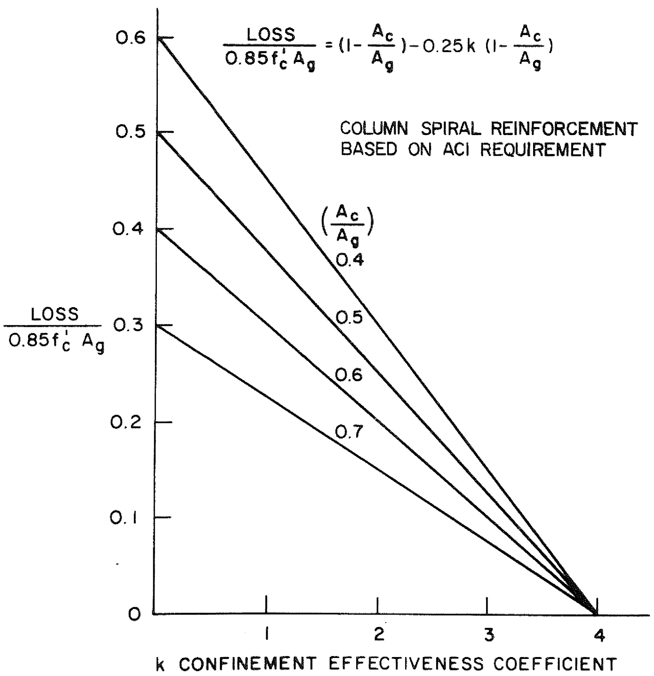


FIGURE 15: LOSS OF COMPRESSIVE STRENGTH DUE TO SPALLING VS. CONFINEMENT EFFECTIVENESS COEFFICIENT

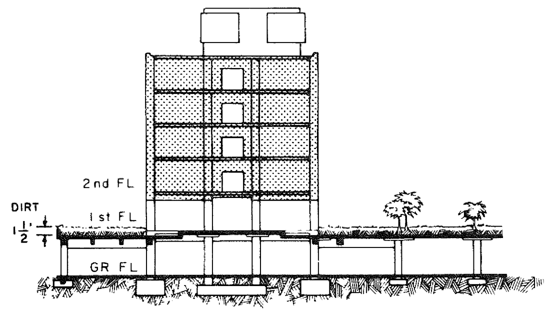


FIGURE 16: ELEVATION OF OLIVE VIEW MAIN TREATMENT BUILDING ILLUSTRATING PRESENCE OF UNNECESSARY MASSES

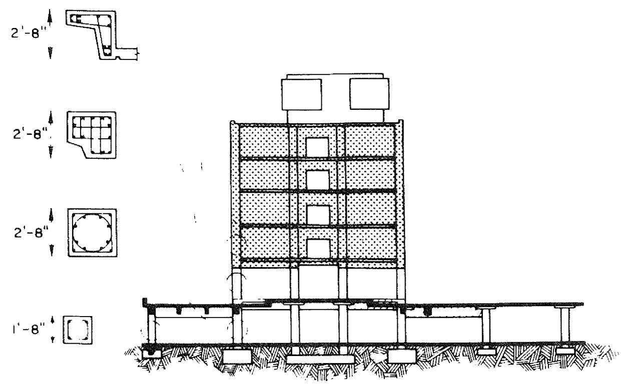
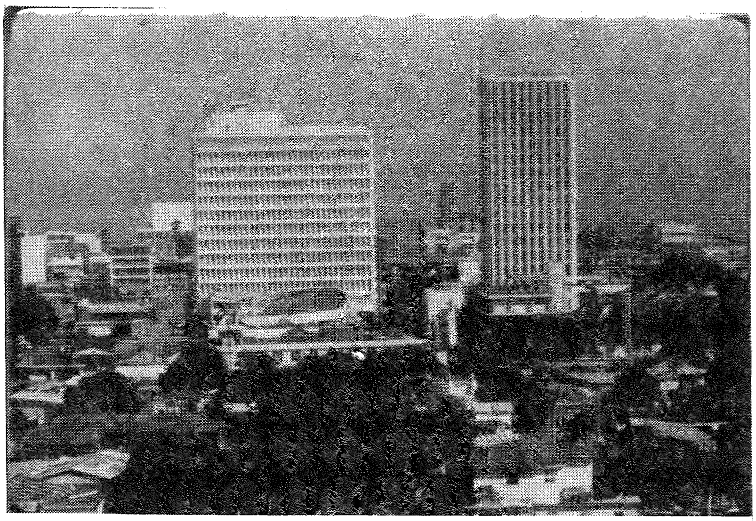
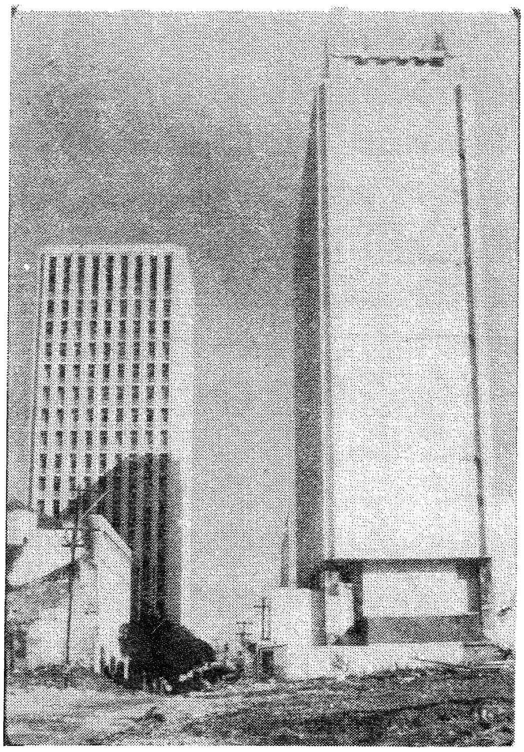


FIGURE 17: ELEVATION OF OLIVE VIEW MAIN TREATMENT BUILDING ILLUSTRATING DISCONTINUING STIFFNESS, STRENGTH, AND DUCTILITY



VIEW OF THE SOUTH FACE



VIEW OF THE WEST FACE

FIGURE 18: VIEWS OF BANCO DE AMERICA AND BANCO CENTRAL, MANAGUA.

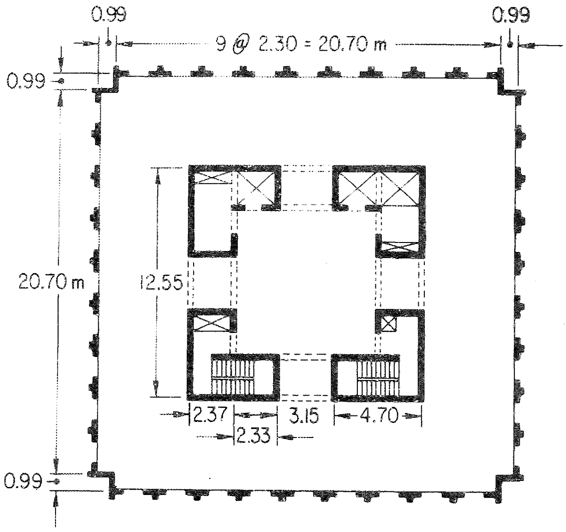


FIGURE 19: PLAN OF TYPICAL FLOOR OF THE TOWER OF BANCO DE AMERICA.

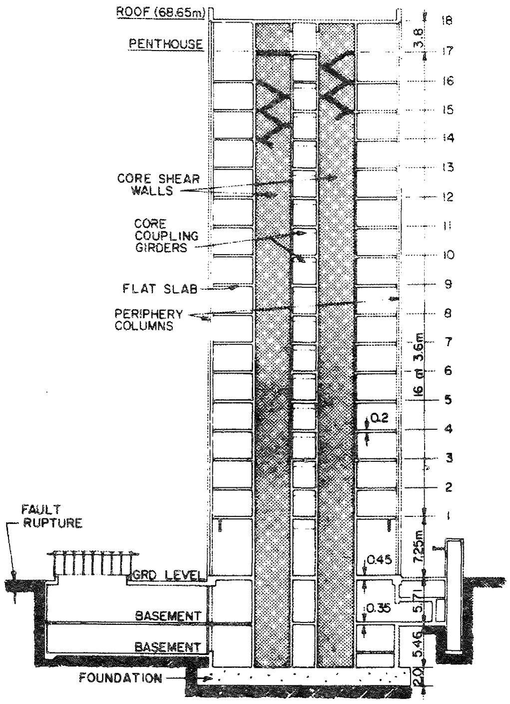
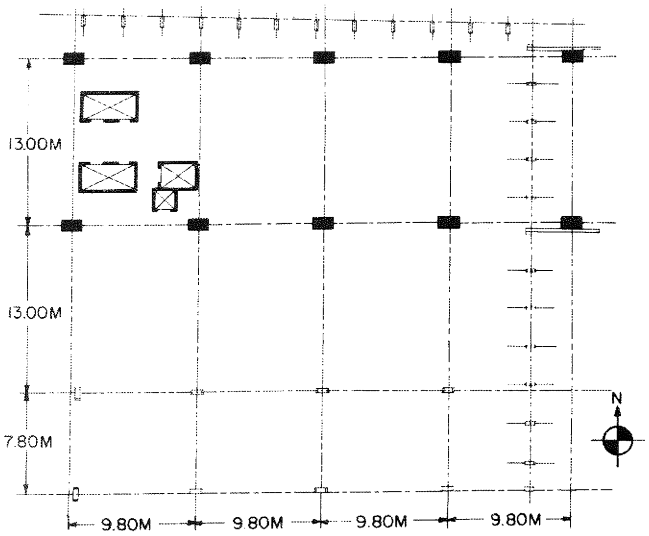
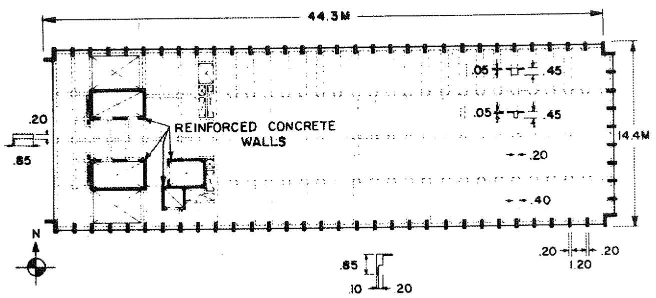


FIGURE 20: ELEVATION OF THE TOWER OF BANCO DE AMERICA.



(a) PLAN OF COLUMNS BELOW 4TH FLOOR



(b) TYPICAL PLAN

FIGURE 21: PLANS OF BANCO CENTRAL

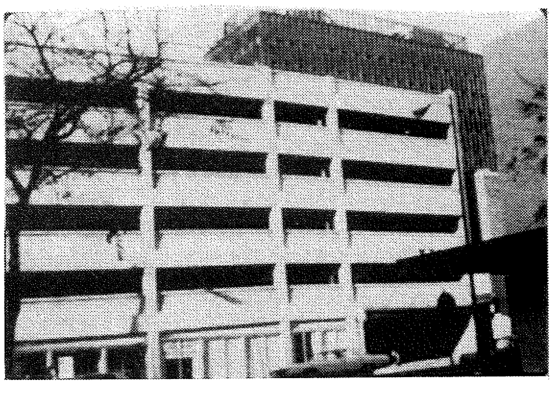


FIGURE 22: SPANDREL-WALL GIRDER - SHORT COLUMN SYSTEM PARKING GARAGE, BERKELEY

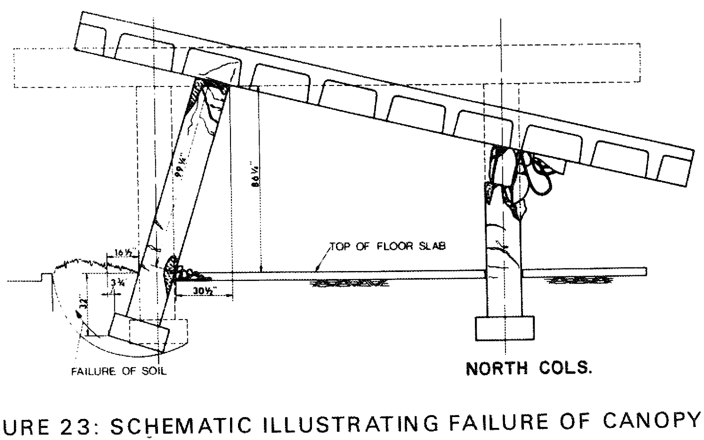


FIGURE 23: SCHEMATIC ILLUSTRATING FAILURE OF CANOPY

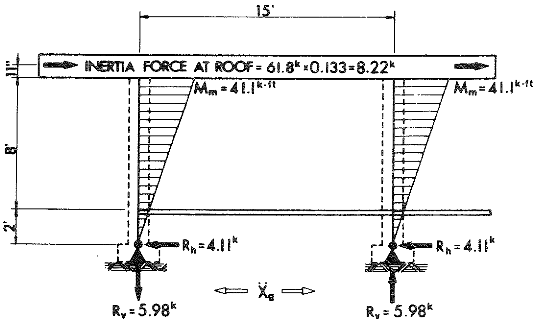


FIGURE 24: DESIGN SEISMIC FORCES AND MOMENTS

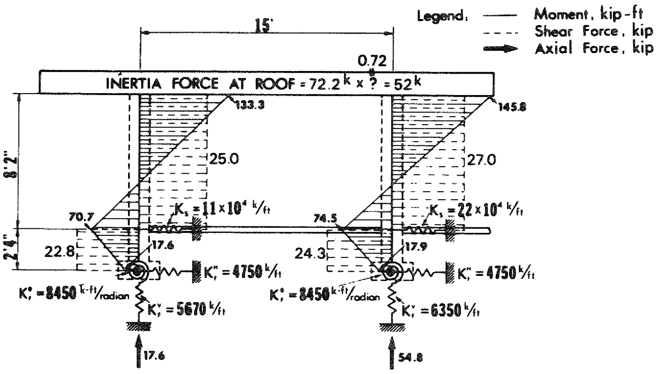


FIGURE 25: MOMENT, AXIAL AND SHEAR FORCES BASED ON REALISTIC STRUCTURAL MODEL

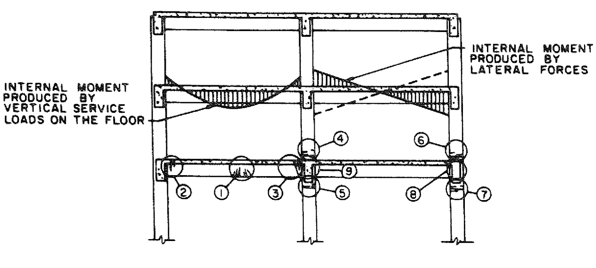


FIGURE 26: CRITICAL REGIONS

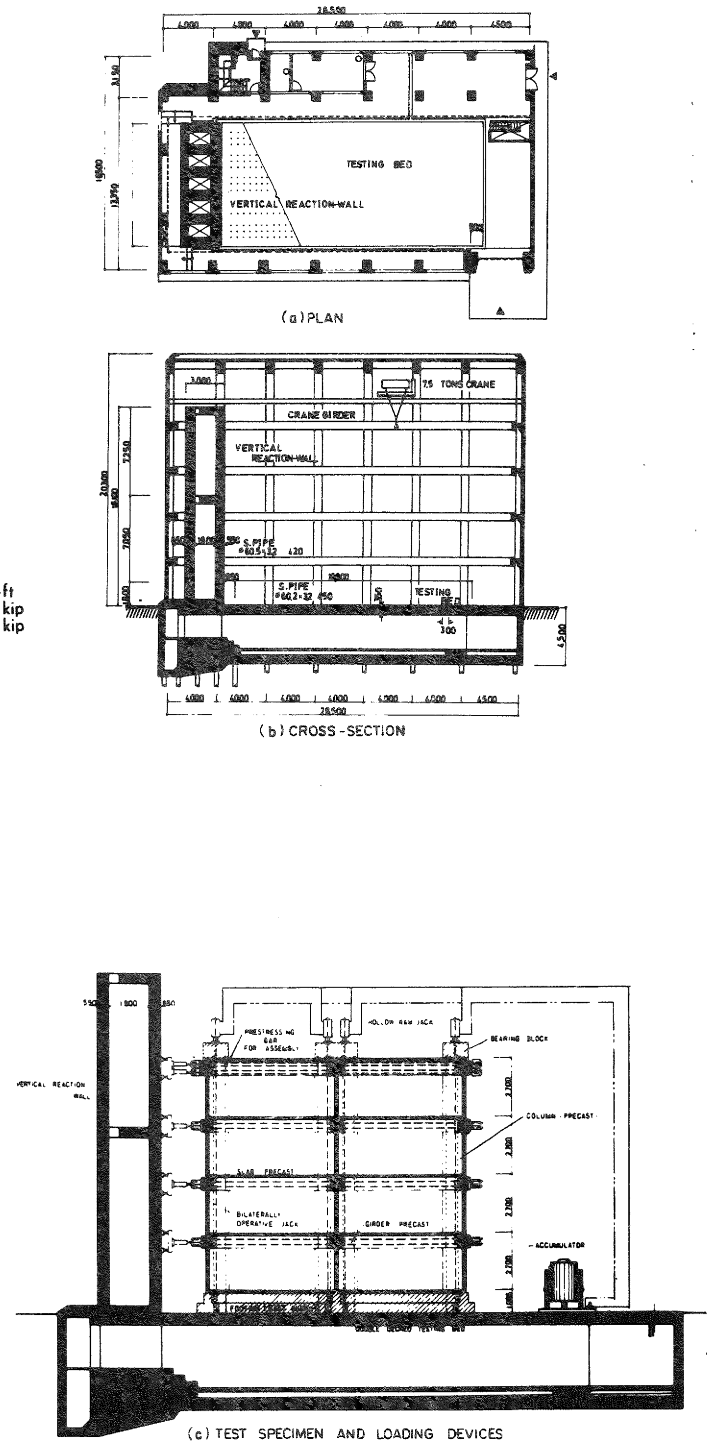


FIGURE 27: LARGE-SCALE PSEUDO-STATIC FACILITY OF THE BUILDING RESEARCH INSTITUTE, TOKYO (46-48)

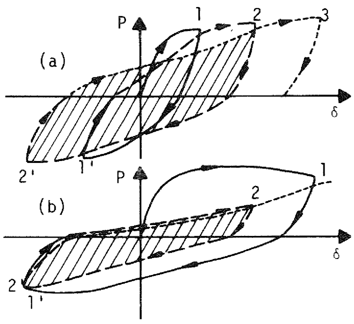


FIGURE 28: EFFECT OF LOADING HISTORY ON FORCE-DEFORMATION RELATIONSHIP

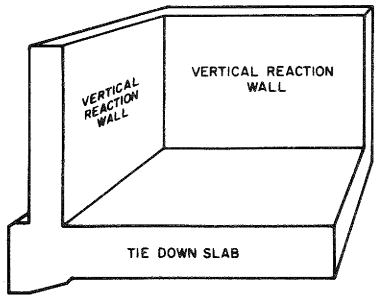


FIGURE 29: PSEUDO-STATIC FACILITY FOR TESTING LARGE-SCALE SPECIMENS UNDER THREE-DIRECTIONAL DEFORMATIONS

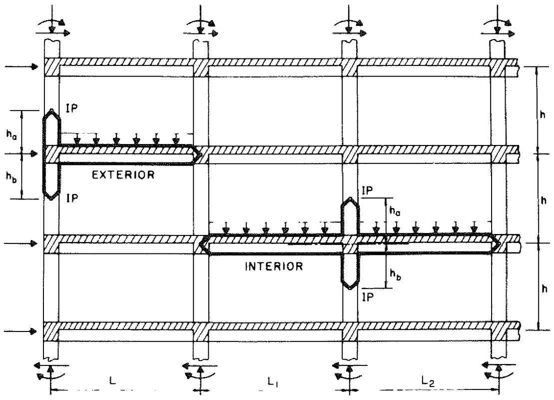


FIGURE 30: BEAM-COLUMN SUBASSEMBLAGES

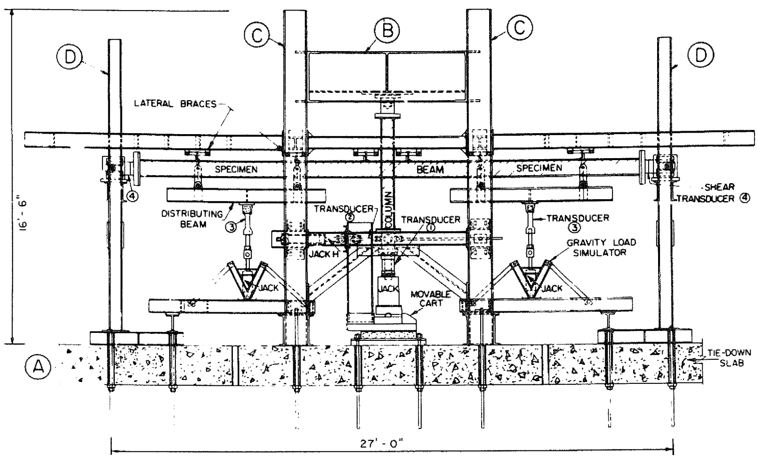
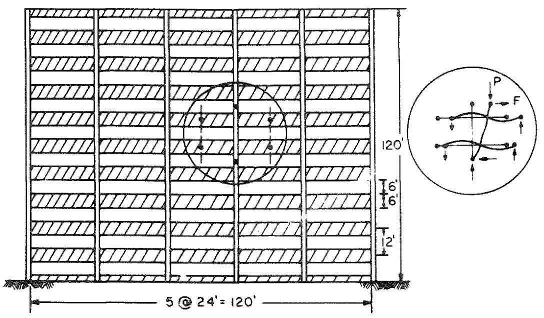


FIGURE 31: TEST SETUP USED FOR TESTING BEAM-COLUMN SUBASSEMBLAGES (51)



SCHEMATIC FRAME, SUBASSEMBLAGE AND LATERAL DISPLACEMENT PATTERN

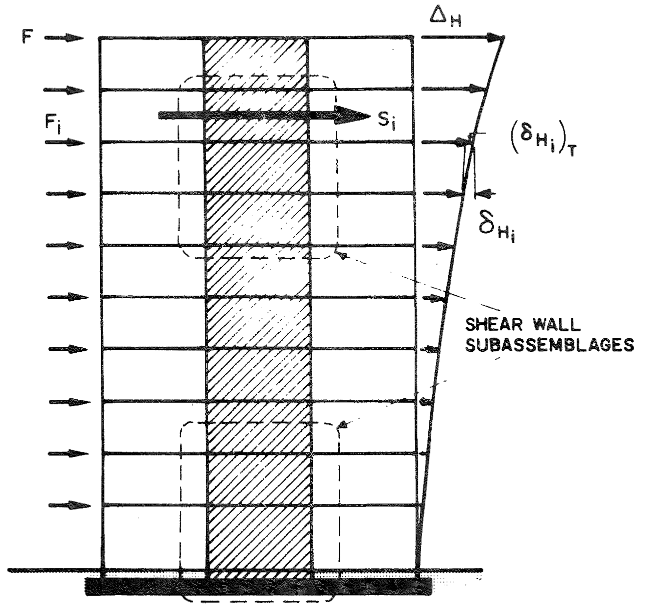


FIGURE 33: HIGH-RISE SHEAR WALL STRUCTURAL SYSTEM

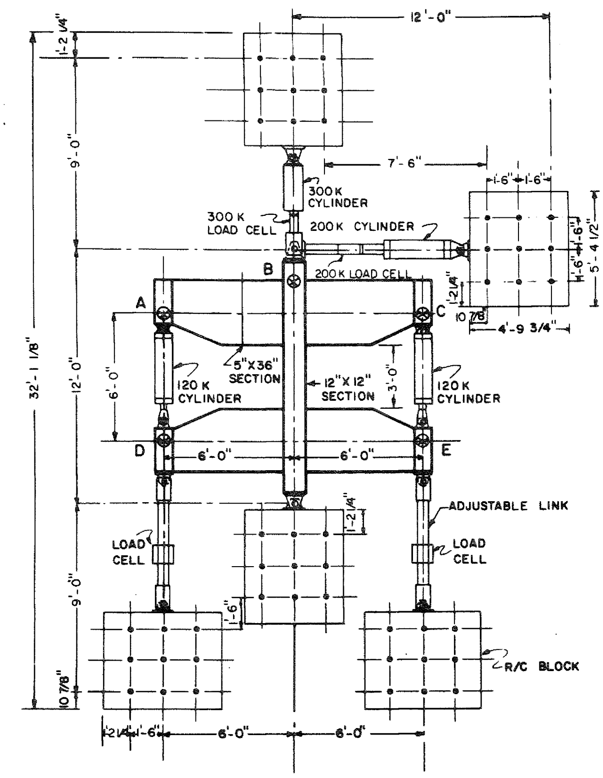


FIGURE 32: GENERAL PLAN OF FACILITY FOR TESTING SUBASSEMBLAGES (52)

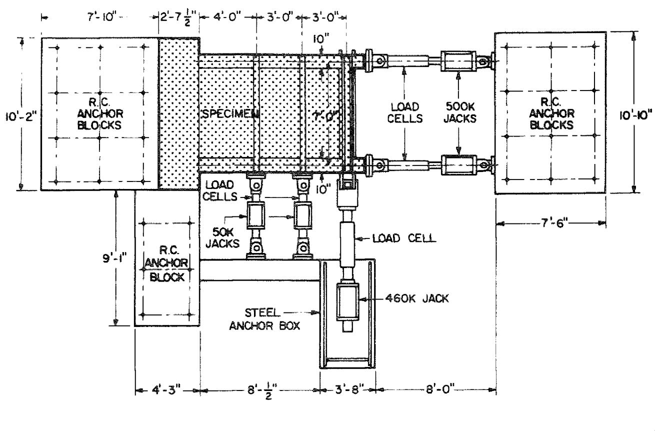


FIGURE 34: GENERAL PLAN OF FACILITY FOR TESTING SHEAR WALL SUBASSEMBLAGES

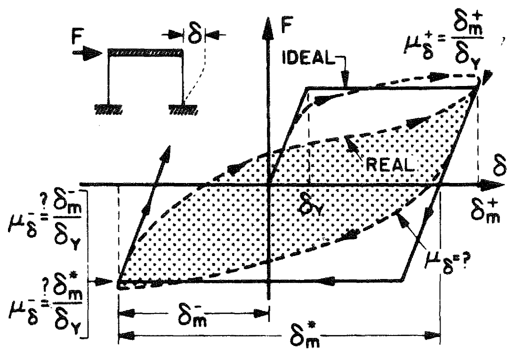


FIGURE 35: DEFINITION OF DUCTILITY FACTORS.

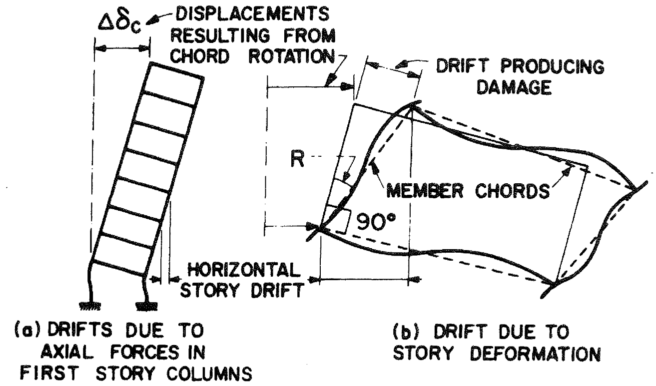


FIGURE 36: SOURCES OF DISPLACEMENTS

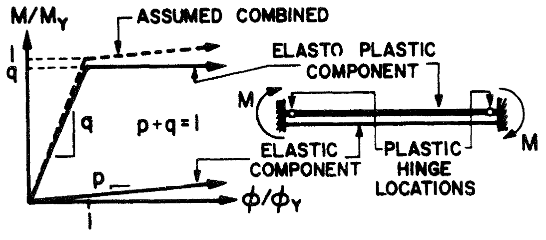


FIGURE 37: TWO-COMPONENT MODEL

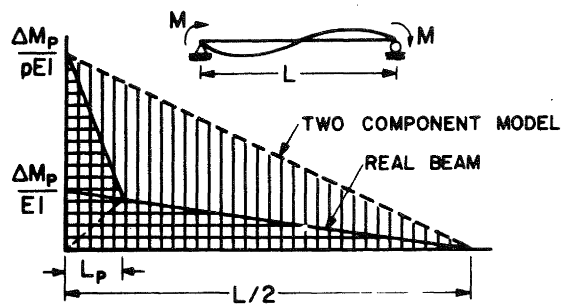


FIGURE 38: INCREASE IN CURVATURES AFTER YIELDING.

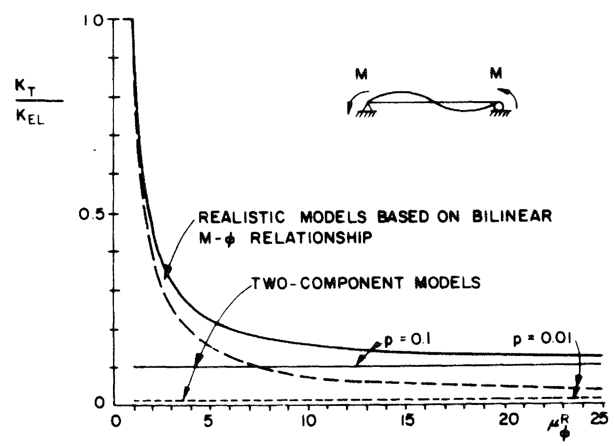


FIGURE 39: TANGENT STIFFNESSES VS. CURVATURE DUCTILITY FACTOR.

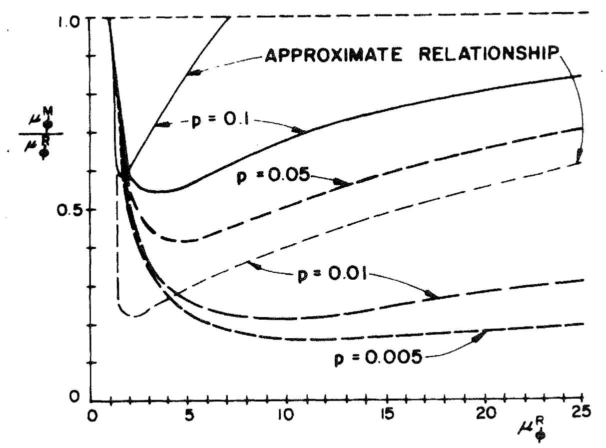


FIGURE 40: VARIATION OF μ_ϕ^M/μ_ϕ^R WITH μ_ϕ^R