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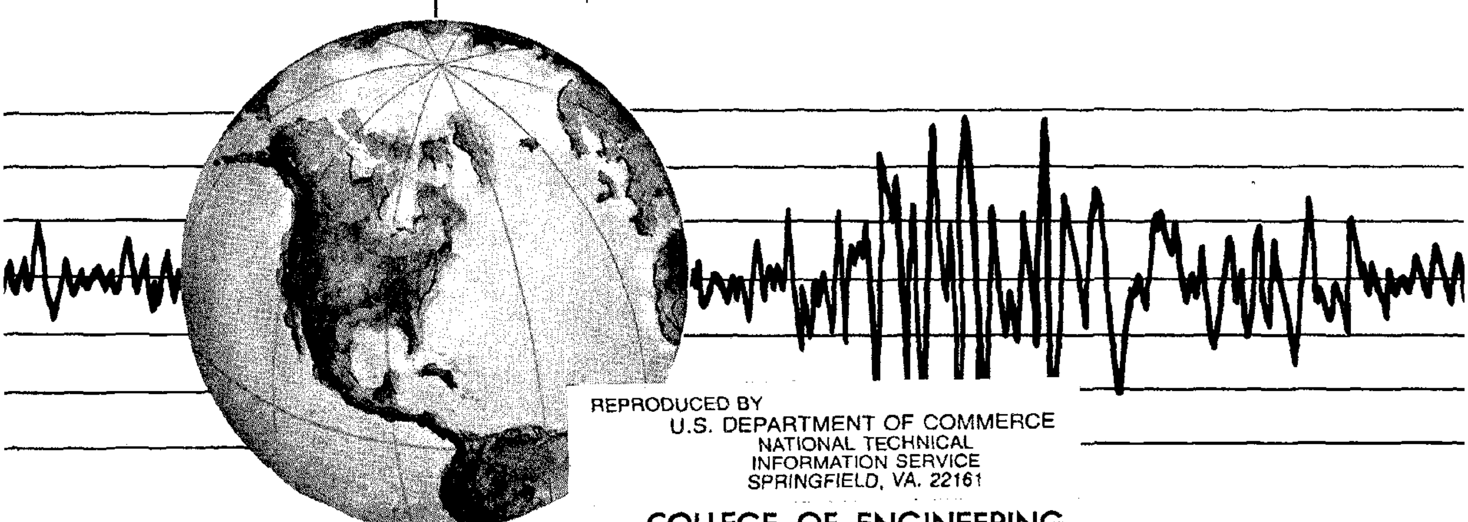
EARTHQUAKE ENGINEERING RESEARCH CENTER

IMPLICATIONS OF RECENT EARTHQUAKES AND RESEARCH ON EARTHQUAKE-RESISTANT DESIGN AND CONSTRUCTION OF BUILDINGS

by

VITELMO V. BERTERO

Report to the National Science Foundation



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Earthquake Engineering Research Center
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ABSTRACT

After an overview of the special problems inherent in the design of earthquake-resistant buildings to be constructed in regions of high seismic risk, the states of the art and practice needed to solve these problems in the U.S. are briefly discussed. Some lessons learned from recent earthquakes, particularly from the earthquakes that occurred in Chile and Mexico in 1985, are discussed as are some results of integrated analytical and experimental research at the University of California, Berkeley. The implications of the ground motions recorded during the 1985 Mexican and Chilean earthquakes, the performance of buildings during the Mexican earthquake, and the research results previously discussed are then assessed with respect to seismic-resistant design regulations presently in enforce as formulated by ATC 3-06 and the *Tentative Lateral Force Requirements* recently formulated by the Seismology Committee of SEAOC. The rationality and reliability of the values suggested by the ATC for the "Response Modification Factor R" and by the SEAOC Seismology Committee for the "Structural Quality Factor R_w" are assessed in detail. The report concludes with general observations and conclusions, and proposes two solutions for the improvement of earthquake-resistant design of building structures: an ideal (rational) method to be implemented in the future, and a compromise solution that can be implemented immediately.

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1. INTRODUCTION

1.1 INTRODUCTORY REMARKS

It is well recognized that most human injury and economic loss due to moderate or severe earthquake ground motions is due to the failure (physical collapse or serious structural and/or nonstructural damage which can jeopardize human life and/or the function of the structure) of civil engineering structures (particularly buildings) many of which were presumed to have been designed and constructed to provide protection against natural hazards. One of the most effective ways to mitigate the destructive effects of earthquakes is to improve existing methods and/or to develop new and better methods of designing, constructing, and maintaining new buildings and of repairing, retrofitting, and maintaining existing buildings.

In an attempt to realize such improvements, the author and his research associates have carried out a series of studies examining the problems encountered in the area of improving earthquake-resistant design and into the development of more reliable approaches to design. The states of the art and practice of earthquake-resistant design and construction of building structures have been reviewed in a series of recent publications by the author and his colleagues [1-6]. The importance of a number of the problems that have been under study and mentioned in these reviews has recently been confirmed by: the ground motions recorded during two major earthquakes recorded during 1985 (the 3 March 1985 earthquake in Chile and that of 19 September 1985 in Mexico); the results obtained from the processing of such records; the performance of buildings during these 1985 earthquakes; and the results of integrated analytical and experimental studies being conducted by the author and his associates in Berkeley.

The author believes that the information obtained and the lessons learned from the above two earthquakes, and the research results of the studies conducted at Berkeley, **are important to the earthquake engineering community**, particularly as the Seismology Committee of the Structural Engineers Association of California (SEAOC) has recommended significant changes [7] in the earthquake-resistant design approach of the SEAOC *Blue Book* [8].

1.2 OBJECTIVES AND SCOPE

The objectives of this report are to present, evaluate, and discuss the importance of some of the information obtained from recent earthquakes and investigations regarding the states of the art and practice in earthquake-resistant design and construction of buildings and to suggest two new approaches to improve such design and construction.

The report begins with an overview of the special problems inherent in the design and construction of earthquake-resistant buildings. The state of the practice in U.S. earthquake-resistant design is then briefly discussed by analyzing the reliability of present U.S. Code seismic-resistant design procedures in light of some of the ground motions recorded during the 1985 Chilean and Mexican earthquakes, and the performance of buildings during these earthquakes. The implications of this building performance and recent research results are then assessed, particularly with regard to the rationale for and reliability of the values suggested by the ATC (NEHRP) [9] for the "Response Modification Factor R" and by the Seismology Committee of SEAOC [7] for the "Structural Quality Factor R_w." Finally, some general observations and conclusions are offered and two solutions for improving the earthquake-resistant design of building structures are suggested: an ideal (rational) solution for the near future, and a compromise solution which can be implemented immediately.

2. OVERVIEW OF SPECIAL PROBLEMS ENCOUNTERED IN DESIGN AND CONSTRUCTION OF EARTHQUAKE-RESISTANT BUILDINGS

2.1 PRELIMINARY REMARKS

While a sound preliminary design of a structure and reliable analyses of this design are necessary, they are not sufficient to ensure a satisfactory earthquake-resistant structure. The seismic response of the structure depends on the state of the whole soil-foundation and superstructure system when earthquake shaking occurs, i.e., response depends not only on how the structure has been constructed, but on how it has been maintained up to the time that the earthquake strikes. A design can only be effective if the model used to engineer the design can be and is constructed and maintained [1, 2]. The authors of reference 10 studied the divergence between building vulnerability and observed damage by applying fuzzy-set theory. They concluded that the variation in quality can change substantially a building's anticipated vulnerability, so much so that observed damage variability may be more easily attributed to quality variations than to inadequacies in engineering design approaches. The authors prescribe a logical approach to decreasing unacceptable and unexpected building earthquake performance: focus on incorporating engineering design penalties for configurations that are ineffective; better supervise the engineering design process; and prescribe better field technical supervision of the construction process. This is *in lieu* of focusing on increasing the levels of engineering requirements for all buildings. Although the importance of construction and maintenance in the seismic performance of structures has been recognized, insufficient effort has been made to improve these practices (e.g., supervision and inspection).

2.2 DIFFERENCES BETWEEN ANALYSIS AND DESIGN, AND BETWEEN DESIGN AND CONSTRUCTION

A preliminary structural design should be available to conduct linear elastic and nonlinear (inelastic) analyses of the soil-foundation-superstructure model(s). To recognize clearly the differences between analysis and design, and at the same time to identify problems inherent in the design of earthquake-resistant structures, it is convenient to analyze the main steps involved in satisfying what can be called the **basic design equation**:

DEMAND	$\leq$	**SUPPLY**
on		of
STIFFNESS STRENGTH STABILITY ENERGY ABSORPTION & ENERGY DISSIPATION CAPACITIES		STIFFNESS STRENGTH STABILITY ENERGY ABSORPTION & ENERGY DISSIPATION CAPACITIES

Evaluation of the **demand** and prediction of the **supply** are not straightforward, particularly for earthquake-resistant buildings. Determination of the **demand**, which usually is done by numerical analyses of mathematical models of the entire soil-foundation-building system, depends on the interaction of this system as a whole and the different excitations that originate from changes in the system environment and of the **intrinsic interrelation between the demand and supply itself** [2].

In the last three decades our ability to analyze mathematical models of buildings when subject to earthquake ground shaking has improved dramatically. Sophisticated computer programs have been developed and used in the numerical analysis of the seismic response of three-dimensional mathematical models of the bare structure of a building to certain assumed earthquake ground motions **(earthquake input)**. In general, however, these analyses have failed to predict the behavior of real buildings, particularly at ultimate limit states. As a consequence of this and due to the lack of reliable models to predict the **supplies to real structures**, there has not been a corresponding improvement in the design of earthquake-resistant structures.

The proportioning (sizing) and detailing of the structural elements of a building are usually done through equations derived from the theory of mechanics of continuous solids or using empirical formulae. Except in the case of pure flexure, a general theory with reliable equations that can accurately predict the energy absorption and dissipation capacities of structural members, and therefore of real buildings, has not been developed.

The three basic elements of the earthquake response problem—earthquake input, demands on the structure, and supply capacity of the structure—are discussed briefly below, together with comments on the importance of proper construction and maintenance.

2.2.1 Earthquake Input: Specification of Design Earthquakes and Design Criteria. The design earthquake depends on the design criteria, i.e. the limit state controlling the design. Conceptually, the design earthquake should be that ground motion that will drive a structure to its critical response. In practice, the application of this simple concept meets with serious difficulties because, first, there are great uncertainties in predicting the main dynamic characteristics of ground motions that have yet to occur at the building site, and, secondly, even the critical response of a specific structural system will vary according to the various limit states that could control the design.

Seismic codes have specified design earthquakes in terms of a building code zone, a site intensity factor, or a peak site acceleration. Reliance on these indices, however, is generally inadequate and methods using **ground motion spectra** (GMS) based on **effective peak acceleration** (EPA) have recently been recommended [1]. While this has been a great improvement conceptually, great uncertainties regarding appropriate values for EPA and GMS persist.

(a) Specifying Effective Peak Accelerations (EPA). The concept of EPA was introduced in the development of zoning maps for ATC 3-06. At first, EPA may appear to be a sound parameter to apply in seismic hazard analysis; however, there is at present no systematic, quantitative definition of this parameter. From results obtained in a recent study [1], it has been concluded that "generally EPA depends both on the type of earthquake considered and the interaction of the dynamic characteristics of the ground motion and of the soil-foundation-superstructure system. Furthermore, EPA will depend on the limit state under consideration. Although the use of EPA can provide an idea of the relative damage potential of a given ground motion, its use as the sole parameter to define this damage potential can be very misleading." In short: "Intensity observations which do not carefully consider the detailed characteristics of the observed structure are likely to be not easily compared." [10]

In developing the ATC design provisions [9], two parameters were used to characterize the intensity of design ground shaking: the EPA (A_a) and the Effective Peak Velocity-Related Acceleration, EPV (A_v). According to ATC, for any specific ground motion, the values of these two parameters can be obtained by the following procedure: (a) the five-percent damped ($\xi = 5\%$) linear elastic pseudo-acceleration spectrum is drawn for the actual given motion; (b) straight lines are fit to the spectral shape for fundamental building periods, T, in the range between 0.1 and 0.5 seconds for the EPA and at a period of about 1 second for the EPV to obtain a smoothed spectrum; and (c) the ordinates of the smoothed spectrum are divided by 2.5 to obtain the EPA and EPV.

Analysis of the five-percent damped linear elastic response spectrum for the recorded ground motion at the Pacoima Dam (or for the derived Pacoima Dam record) shows that the maximum ATC-specified values for EPA and EPV, namely $A_a = 0.40$ and $A_v = 0.40$, can be significantly exceeded for certain period values in the range used for their derivation. Nevertheless, the recommendation of these values by ATC has been a welcome step towards a more realistic appraisal of the severity of the ground motion that can occur at sites located in the proximity of major active faults (map area no. 7 of the ATC maps).

2.2.2 Estimation of Reliable Demands. The major uncertainties in the estimation of reliable demands, usually obtained by numerical analysis, are due to **difficulties in predicting the following**: (1) the critical seismic loading during the service life of the structure (properly established design earthquakes); (2) the state of the entire soil-foundation-building system when the critical ground motion occurs at the site of the building (proper selection of the mathematical model(s) to be analyzed); (3) the internal forces (deformation) and stresses (strains) that will be induced in the model (structural and stress analysis); and (4) realistic supplies of stiffness, strength, stability, and energy absorption and energy dissipation capacities (i.e. realistic hysteretic behavior) of the entire soil-foundation-building system.

2.2.3 Prediction of Supplies. The supplies to a building depend not only on the supplies to its bare superstructural system, but also on the supplies that result from the interaction of the bare superstructural system with the soil-foundation and the so-called **nonstructural components** of the building. For example, masonry walls and/or partitions tightly packed as infill into the moment-resisting frames of a building introduce significant changes in the dynamic characteristics of that building. Changes in stiffness, strength, and deformation capacities are illustrated in Figure 1. An evaluation of the test results illustrated in this figure and implications of these results for the design of earthquake-resistant buildings are discussed in reference 11. It is obvious that when such interaction occurs between structural and nonstructural components, **neglecting such interaction in the selection of numerical characteristics for and of the design of the structure can lead to evaluation of demands that are completely unrealistic and, consequently, to a poor final design of the entire building system.**

In considering the general design equation, the designer might be tempted to overcome the problems created by the uncertainties to which the values of the **demands** are subject by increasing the **supplies**. However, an increase in supply must be done very carefully because it may considerably increase the demand.

2.2.4 Proper Construction and Maintenance of Buildings. Design and construction are intrinsically interrelated—if good workmanship is to be achieved, the detailing of members and of their connections and supports must be simple. As noted in the preliminary remarks to this report, a design is only effective if it can be realized in construction and is properly maintained. Field inspection has revealed that a great deal of damage and failure has been due to poor quality control of structural materials and/or poor workmanship—**problems that would not have arisen if the building had been carefully inspected during construction.** In many other cases, damage may be attributed to **improper maintenance of buildings during their service lives.** Inappropriate alteration, repair, and/or retrofitting of the structure, as well as of nonstructural components, can lead to severe damage following major earthquake shaking.

2.3 SUMMARY

The above review of the problems encountered in achieving effective earthquake-resistant construction of buildings clearly indicates the need for a comprehensive approach to these problems, an approach in which the various disciplines involved in the design, construction, and maintenance of earthquake-resistant buildings are integrated. The need for such an approach has been discussed in reference 1: the ultimate goal should be a sound seismic design code procedure which would be both simple enough to facilitate the preliminary design of a building and yet ensure capable inspections during all phases of design, construction, and maintenance (modifications, repair and/or retrofitting).

3. STATE OF PRACTICE OF EARTHQUAKE-RESISTANT DESIGN

3.1 INTRODUCTORY REMARKS

Since the design and construction of most earthquake-resistant buildings will, in practice, generally follow seismic code provisions, it is convenient to examine these provisions briefly, and to examine what has been done and what should be done to improve the present state of the practice.

3.2 ESTIMATION OF DEMANDS IN PRESENT U.S. SEISMIC CODES

There are several sources of uncertainty in the estimation of demands, uncertainties that can be grouped in two categories: (1) specified seismic forces; and (2) methods used to estimate response to these seismic forces.

3.2.1 Estimation of Seismic Forces. For regular buildings, the lateral seismic forces can be derived as follows.

(a) Base Shear:

$$V = C_s\, W = \frac{C_{sp}}{R}\, W \tag{1}$$

where V is base shear, C_s is defined as the design seismic coefficient, W is the weight of the reactive mass (i.e., the mass that can induce inertial forces), C_{sp} is the seismic coefficient equivalent to a linear elastic response spectral acceleration, S_a, ($C_{sp} = C_s\, R = \frac{S_a}{g}$), and R is the reduction factor.

(b) Distribution of Base Shear Over Height of Superstructure:

$$V = F_t + \sum_{i=1}^{n} F_i \tag{2}$$

where F_t is concentrated force at the top and represents the effects of higher modes (whiplash effect) and:

$$F_i = \frac{(V - F_t)\, w_i h_i}{\sum_{i=1}^{n} w_i h_i} \tag{3}$$

is the force at level i (usually at the floor level), w_i is the portion of W located at or assigned to level i, and h_i is the height above the base to level i.

3.2.2 Estimation of Structural Response to Seismic Forces. Structural response can be estimated using linear elastic analyses, directly using the above statically equivalent lateral forces (equations 2 and 3) or these forces multiplied by load factors depending on whether the design will be performed using allowable (service) stress or the strength method.

The uncertainties involved in the estimation of base shear and its distribution over the height of the structure as well as the reliability of the procedures and values specified by present U.S. seismic codes will be discussed below. Before doing so, however, it is convenient to discuss the state of the art of specifying design criteria and design earthquakes.

3.3 STATE OF ART OF SPECIFYING DESIGN CRITERIA AND DESIGN EARTHQUAKES

Having discussed the special problems encountered in the design of earthquake-resistant buildings, it is appropriate to describe the state of the art with respect to each of these problem areas, following the same sequence. This has been done in references 1-4 and 6. Here, only the state of the art of the first and most difficult step in the design process is considered—specification of the **design earthquake** (EQ)—giving due consideration to the three main limit states that may be specified with regard to building response: serviceability level—where the building is expected to continue to perform its designated function; damageability level—where the damage is limited to predetermined levels; and safety against collapse—where any degree of damage that will not endanger human life is permitted. When the appropriate design criterion has been selected, the design earthquake is defined according to the following guidelines.

3.3.1 Design Earthquake (EQ) for Serviceability Limit States. For all practical purposes, the building should remain in the linear elastic state. While a design EQ based on a smoothed linear elastic design response spectrum (LEDRS) is the most reliable and convenient approach for the preliminary design, the ground spectrum that is used to derive the LEDRS must be appropriate to the site and not based just on standard values. Values selected for the damping ratio, determination of allowable stresses, and computation of natural periods and internal forces must be consistent with expected behavior. This was the approach followed by ATC 3-06 in defining the recommended spectral shapes for deriving LEDRS [9].

3.3.2 Design Earthquake (EQ) for Ultimate Limit States (Damageability and Safety Against Collapse). Derivation of a reliable inelastic design response spectrum (IDRS) requires full characterization of the expected severe ground motions at the site as well as acceptable structural responses. However, current methods used to calculate IDRS do not account for the duration of strong ground shaking. Extensive integrated analytical and experimental studies will be required to obtain the information necessary to establish reliable design earthquakes when ultimate limit states control the design. Until this is done, the procedure suggested in references 1 and 2 can be used. This procedure requires the derivations of inelastic response spectra corresponding to the available recorded ground motions through nonlinear dynamic time history analyses of structures with different degrees of ductility ratio.

3.4 RELIABILITY OF SEISMIC CODE PROCEDURES FOR DETERMINING VALUES FOR BASE SHEAR

As indicated previously in discussing the estimation of demands prescribed by present U.S. seismic codes, a topic discussed in more detail in references 1 and 2, the determination of seismic forces is typically conducted according to equations (1) through (3). There are several sources of uncertainty, however, in estimating the values of these seismic forces, some of which are discussed below.

As indicated by equation (1), C_s and W must be estimated before V can be determined. Although the uncertainties involved in estimating C_s are more substantial than those involved in estimating W, there are nonetheless difficulties in estimating the latter value accurately.

3.4.1 Estimation of Seismic Reactive Weight. Conceptually, W should equal the weight of the reactive mass of the building, i.e. **the weight of the mass that can give rise to inertial forces.** The UBC, ATC, and the 1985 SEAOC define this as the total dead load and applicable portions of other loads that are listed separately. "In storage and warehouse occupancies, a minimum of 25 percent of the floor live load should be applicable." Why only 25%? There are cases in which most of the mass of the live load can react, i.e. develop inertial forces. Thus, the designer should carefully ascertain the live load that could act on a building during structural response before adopting only the recommended 25% of the specified live load. A main reason for the failure (collapse) of several buildings in Mexico City during the 1985 earthquake was the very heavy live loads (heavier than previously estimated and specified in the codes) which acted as reactive masses during the earthquake.

3.4.2 Estimation of Code Values for C_s. The value specified in seismic codes for C_s depends on several factors. In general, however, it is possible to distinguish three main parameters. First, C_s depends on a derived smoothed LEDRS, i.e. on $C_{sp} = S_a/g$ (some codes base this design response spectra on pseudo-acceleration, while others on absolute acceleration) for the seismic zone on which the building is to be constructed. Secondly, it depends on what can be called the reduction factor (R) for the required linear elastic strength to obtain what can be considered the IDRS and this reduction factor is usually based on the expected available ductility of the designed structure. In turn, the value of R depends on the design method, i.e. allowable stresses or strength (yielding or ultimate) methods. Thirdly, it depends on the estimation of the fundamental period, T, of the building.

The information available for reliable determination (estimation) of these three parameters is scant. There is a need for instrumenting thoroughly regions of high seismic risk as well as buildings located in these regions. A brief discussion of the uncertainties involved in specifying these parameters follows.

3.5 RELIABILITY OF SEISMIC CODE PROCEDURES FOR DETERMINING LINEAR ELASTIC DESIGN RESPONSE SPECTRA, LEDRS

Due to insufficient reliable (measured) data on earthquake ground motions, the formulation of design spectra is currently based on inadequate statistical information. The information obtained from the records of the severe ground motions that developed in some of the earthquakes of the last fourteen years have altered the previous statistical base so dramatically that drastic changes in the design response spectra and therefore in the code-specified C_s have been required. Examples of these are the records obtained from: the 1971 San Fernando earthquake; the 1979 Imperial Valley earthquake; and the recent 1985 Chilean and Mexican earthquakes, the latter being perhaps the most dramatic as will be discussed below.

Until 1971, the recorded NS component of the 1940 El Centro earthquake was considered the most extreme earthquake ground motion that could be expected. The records obtained during the 1971 San Fernando Valley earthquake demonstrated, however, that the damage potential of this El Centro component was very low compared with that of the recorded San Fernando motion.

3.5.1 The 19 September 1985 Mexican Earthquake.

(a) Mexico City Ground Motions and Response Spectra. According to available statistical information (which was based on recorded and estimated ground motions resulting in a maximum estimated value for the peak ground acceleration of 50 cm/sec^2), the 1976 seismic code provisions for the Mexico Federal District (Distrito Federal) was based on the 5% damped linear elastic design response spectra (LEDRS) shown in Figure 2a. The ground motions recorded in Mexico City, located 400 km from the epicentral region, during the 1985 earthquake show very different intensities depending on the soil condition at the location of the recorder. The acceleration records and the 5% damped linear elastic response spectra for the absolute acceleration corresponding to horizontal components of the acceleration recorded by accelerographs located in the Ciudad Universitaria (CU) (on firm soil—zone I) and by those located in the Centro SCOP of the Secretary of Communication and Transportation (SCT) (on the highly compressible soil of zone III) are shown in Figure 3a. Comparison of these response spectra clearly indicates the importance of soil conditions (soil profile) to the ground motion at the free-field surface. While the maximum ground acceleration recorded at CU was 39 cm/sec^2, the EW component recorded at SCT has a peak value of 168 cm/sec^2, i.e. a difference of more than a factor of four.

The importance of the recorded ground motions with respect to C_s becomes evident when their 5% damped linear elastic response spectra (LERS) (Figure 3b) are compared with those on which the derivation of the C_s was based (Figure 2a). This comparison shows that for group B buildings located in zone III, the 5% damped LERS of the ground motion recorded at SCT exceeds the design response spectra assumed by the code for all structures having fundamental periods up to 3.2 seconds. For structures with fundamental periods of about 2 seconds, the response spectra value of the recorded motion is more than four times that adopted by the 1976 Mexico Federal District code.

Another significant result obtained from the motions recorded during the 1985 Mexican earthquake is the resulting spectrum amplification factors. For a period of about 2 seconds, the 5% critical damping spectrum amplification factors for the maximum ground acceleration was 983/168 = 5.85, that is, a value significantly higher than those that have been suggested and significantly higher than are at present used in the U.S. [12]. For example, for a level of probability of One Sigma and 5% critical damping, the amplification factor suggested for the acceleration is 2.71. Therefore, if ground motions similar to those recorded in Mexico City can occur in the U.S., the results discussed above indicate the need for the revision of procedures presently used to develop design response spectra.

It should be noted that because these recorded ground motions were so high, and because so many buildings performed poorly, the 1976 code for the Federal District has already been revised, and a new emergency code based on significantly more intense response spectra has been enforced (Figure 2b). For group A buildings, including essential facilities, the LEDRS in this emergency code is 1.5 times that for ordinary occupancy structures (group B). On the other hand, the 1985 SEAOC tentative requirements specify I to be 1.25, i.e. 1.2 times smaller than the new Mexican requirement.

From comparisons of the 5% damped LERS from the recorded EW component of the ground motion at SCT in Mexico[1] with the 5% damped LEDRS on

[1] A somewhat more demanding response spectra is obtained if the ground acceleration resulting from the combination of the two components NS and EW recorded at SCT is considered. This combination results in an acceleration ground motion of 196 cm/sec^2 in the S 60 E direction.

which the ATC [9] and the 1985 SEAOC recommendations are based (Figure 4), even for the region of highest seismic risk in the United States (ATC map area no. 7 and SEAOC zone 4), the recommended seismic lateral force coefficients C_{sp} (which are based on an assumed 5% damped LEDRS) for the soft clay profiles (soil type S_3) and for periods $1.7 < T < 3$ seconds are significantly smaller than the values corresponding to a similar spectrum obtained from the ground motion recorded at SCT in Mexico City. Thus, the results from the Mexico City records should be carefully assessed regarding our present seismic design response spectra for zones having soil profiles that could be similar in nature to that in the center of Mexico City.

Besides the above lessons learned from or re-emphasized by the 1985 Mexican earthquake, there were many others, among which the following three deserve special attention in the U.S. insofar as seismic-resistant design and construction practice are concerned: (1) the duration of strong motion; (2) soil-building resonance (interaction); and (3) separation of adjacent buildings.

(b) Duration of Strong Motion. As can be seen from the 5% damped LERS of the ground motions recorded at station SCT and that computed from these recorded motions (Figure 3b), for $T = 0.5$ and 0.7 seconds, S_a is nearly 0.3, and increases for T equal to or greater than 1 second, reaching a value of 1 for T approximately equal to 2 seconds. Furthermore, from analysis of this recorded ground motion (Figure 3a), it is clear that the strong motion (say, acceleration $a \geq$ 50 gals) lasted for more than 30 seconds with 9 cycles of reversals exceeding 100 gals. There is no doubt that in structures for which $T \geq$ 0.5 seconds and that were designed according to the Mexican (DF) Code (where C_s is specified to be less than or equal to 0.06), severe oscillations occurred inducing many cycles of reversal yielding which not only caused the stiffness of such buildings to deteriorate significantly (thereby elongating their periods), but

also could have caused their axial-flexural shear, torsional and bond (anchorage) maximum strengths to deteriorate significantly, particularly in the case of reinforced concrete moment-resisting space frames with just waffle slabs or flat plates as floor systems.

This is the first time that recorded ground motions and the corresponding responses of buildings have shown that there is the possibility that code-designed structures can undergo significant numbers of yielding reversals with high ductility ratio demands, considerably larger than was considered possible before the 1985 Mexican earthquake. A large number of buildings in Mexico failed due to this duration of strong motion. It was also observed that there were many foundation (pile) failures, and that several buildings remained inclined and a number overturned. This may have been a consequence of the degradation of the foundation (particularly in the effective friction of friction piles) due to the large number of reversals, with large deformations. Present seismic codes in the U.S. should be reviewed thoroughly with respect to the seismic-resistant design of foundations on poor soil conditions to determine whether this problem has been properly addressed.

(c) Soil-Building Resonance. Most of the buildings that collapsed or suffered severe damage were flexible buildings with initial fundamental $T \geq 0.7$ seconds and whose period increased (lengthened) with the damage that accumulated as a consequence of the response to the long duration of strong motion. Older buildings, with lateral elastic strength of the order of $0.20W$ and very rigid, i.e. with short T, say $T < 0.5$ seconds, located just beside collapsed or seriously damaged flexible buildings, survived the earthquake without serious damage.[2] This is not surprising and is in agreement with a well-known conceptual principle or guideline of seismic-resistant design of structures: use stiff

[2] Some buildings with $T \leq 0.5$ seconds collapsed because their lateral elastic strength was only that or less than that required by the code.

structures on soft soil deposits and flexible structures on hard (firm) soil. This concept is reflected and red-flagged in present UBC seismic regulations through the use of the so-called numerical coefficient for site-structure resonance, S. The value of S varies from 1 to 1.5, the maximum value to be used when the ratio between T and the characteristic site period T_s is equal to 1, i.e. when the building is in resonance with the soil. Although it is possible to argue that the UBC values for S are not adequate (they could be larger or smaller), what is important is the **concept** that is intended by the requirement that T/T_s be computed, i.e. that for the designer to realize an economical design, it is necessary to avoid enhanced seismic response (forces and/or deformations) that can occur when T/T_s tends to 1, i.e. a soil-structure resonance is attained.

It is for the same reason that it is very difficult to justify the changes recommended by SEAOC [7] regarding the evaluation of S. The current UBC requirement that relates the coefficient S for site-structure resonance to the ratio of T/T_s would be eliminated by SEAOC, and S would be related solely to the characteristic of the soil. No attention is paid in the recommended code or red-flagged to the designer regarding the importance of avoiding insofar as is possible designs and/or constructions of flexible structures on soft soil or a stiff building on firm soil. This change is a step backward in the attempt to improve seismic-resistant construction.

(d) Separation of Adjacent Buildings. Many buildings in Mexico suffered serious damage due to the hammering of adjacent buildings due to the lack of proper separation, and this despite the significantly stricter requirement of the 1976 Mexican Code than that of the current UBC. The limitation of 0.005 specified by the UBC on the interstory drift computed through an elastic analysis is unrealistically low. The building separations recommended by the UBC are consequently also inadequate. The ATC has already recognized this

inadequacy and has recommended that the lateral deflection induced by the specified design seismic forces, and determined through an elastic analysis and considering the building to be fixed at the base, be increased by multiplying these seismic forces by a deflection amplification factor C_d. Although the 1985 SEAOC specifies a story drift limitation similar to that of the UBC ($0.04/R_w \leq 0.005$), it is further recommended that separations between adjacent buildings should allow for $3/8\ R_w$ times the displacement due to the design seismic forces. The rationale for this choice of amplification factor is not clear, since it appears that structures designed just to comply with the limitation of $0.04/R_w$ under specified design seismic forces will undergo deflections larger than $3/8\ R_w$ times the displacement under the specified design forces when subjected to major earthquake ground shaking.

A revision of UBC regulations regarding building separation is urgently needed. To avoid the effects of hammering of adjacent tall buildings, separation would be required that could lead to serious problems in the economical use of usually very expensive real estate. Thus it appears that to avoid damage between adjacent buildings it will be necessary to develop other regulations or requirements than just to specify adequate separation, such as including in the design and detailing of adjacent buildings the possibility of such hammering. One such regulation should be that for two adjacent buildings, with inadequate separation, the floor systems of the two buildings should be at the same level.

The problem of proper separation between adjacent buildings urgently requires consideration in our codes. Economical solutions for retrofitting existing adjacent buildings which do not have adequate separation should be researched immediately.

3.5.2 The 3 March 1985 Chilean Earthquake. This earthquake, with an epicenter located in the Pacific Ocean near the coast of the center of Chile, was reported to be of Richter magnitude 7.8. The earthquake ground motions were measured by at least thirty-five strong motion instruments (accelerographs). From evaluation of these records, Saragoni and his associates [13] have concluded that this event can be considered to have consisted of two successive shocks: the first of Richter magnitude 5.3 with a duration of strong motion of 10 seconds, and the second, which occurred 10 seconds later, of Richter magnitude 7.8 with a strong motion duration of about 30 seconds. The duration of recorded motion was in some cases 120 seconds.

The maximum accelerations (peak ground accelerations, PGA) were: 0.67g in the NS direction and 0.60g in the EW direction for the horizontal components measured at Melipilla; and an 0.85g vertical component with 0.67g and 0.43g, respectively, for the N10E and S80E components of PGA measured at LLolleo. The very large vertical components of the ground acceleration recorded at LLolleo deserve special study regarding the possible effect of such ground motion on the response of buildings. The record of the horizontal component in the N10E direction recorded at LLolleo is considered to have the greatest damage potential, with strong motion for nearly 50 seconds. Analysis of the 5% damped spectrum for this component reveals that if a ground motion similar to this component were to occur in the U.S., the maximum value of EPA recommended by ATC, i.e. A_a = 0.40g (considering an amplification factor of 2.5), could be significantly exceeded for periods between 0.1 and 0.8 seconds.

As in the case of the Mexican earthquake, it is of interest to compare the 5% damped linear elastic response spectrum resulting from the recorded N10E component of the ground motion measured at LLolleo with those corresponding to the ATC recommended seismic force coefficients C_{sp} as well as to 1985 SEAOC recommended ZC (equivalent to C_{sp}) values for different soil types. As has already been pointed out, the values of C_{sp} are based on assumed 5% linear elastic response spectra. Such comparisons are illustrated in Figure 5. Even for the region of highest seismic risk in the U.S. (ATC map area no. 7 and SEAOC seismic zone 4), the recommended spectrum for C_{sp} for soil type S_1 (rock and stiff soil) and for periods less than 1.8 seconds is significantly smaller than the values corresponding to the similar spectrum obtained from the N10E ground motion recorded at the LLolleo station. Therefore, the ground motion of the 1985 Chilean earthquake should be carefully evaluated regarding present seismic design spectra as recommended in U.S. codes. The damage potential of the recorded ground motion at LLolleo is significantly greater than any previously recorded or considered by any code for rigid buildings located on stiff soil sites.

Analysis of the recorded ground motions and the computed linear elastic response spectra reveals that the spectrum amplification factors are also somewhat higher than present U.S.-recommended values [12]. For a level of probability of One Sigma and for 5% critical damping, the amplification factor suggested for the acceleration is 2.71, while the maximum recorded amplification factor was 3.6.

3.6 INELASTIC DESIGN RESPONSE SPECTRA: RATIONALITY AND RELIABILITY OF VALUES SUGGESTED FOR REDUCTION OR MODIFICATION RESPONSE FACTORS

As previously discussed, ATC has recommended that values of C_s be derived from a smoothed 5% damped LEDRS which recognizes the severity of the earthquake ground motion that can be anticipated in various seismic zones of the U.S. Although the values recommended for the LEDRS might not be conservative enough for certain regions, as demonstrated by the computed 5% damped LERS for the ground motion recorded during the 1971 San Fernando and 1985

Chilean and Mexican earthquakes, the ATC recommendation has been a welcome step towards a more realistic appraisal of the severity of ground motions that can be anticipated at a given site.

Therefore, the approach followed by ATC 3-06 in recommending design earthquakes in the form of smoothed LEDRS appears to be the correct approach for the design of essential facilities that should remain essentially undamaged even for the **maximum credible earthquake** (MCEQ). However, except for these essential facilities, it would be unrealistically conservative and uneconomical to design most building structures to respond to MCEQ shaking at the site within the linear elastic range of the structural material, or even in the so-called effective linear elastic range of behavior of the structure (i.e., to its significant yield level). As already described by the engineers who developed the original SEAOC *Blue Book* and by those who developed the ATC 3-06 recommendations, in order to realize economical design of buildings that could be subjected during their service life to MCEQ shaking, significant but controllable (acceptable) inelastic deformations of such buildings must be accepted. These inelastic deformations **usually** allow the required linear elastic strength to be reduced without the maximum resulting deformations increasing significantly.

Apparently, however, it is not well recognized that even for a given structural system, the acceptable decrease in the above strength cannot be constant for the entire range of the fundamental period T for which the structure can be designed. In the ATC recommendations, the beneficial effect of inelastic deformation in reducing the required strengths is introduced by means of a **response modification factor** R which is independent of T.

Apparently, the SEAOC Seismology Committee has adopted LEDRS similar to those recommended by ATC by proposing that the numerical coefficient C in the present revision of the Blue Book [7] be defined as follows:

$$C = \frac{1.25\,S}{T^{2/3}}$$

Note that according to this 1985 SEAOC revision:

$$V = \frac{ZIC}{R_w} W$$

Therefore, according to the notation in equation (1), $C_{sp} = ZCI$. To consider the effect of energy dissipation through inelastic deformation, the SEAOC Committee proposes that a numerical coefficient R_w (termed the **structural quality factor** in the final draft of the predecessor to reference 7) be used to reduce C. The only apparent difference in the definition of R and R_w is the level of the design forces to which the LEDRS are reduced. ATC reduces the LEDRS to the "significant yield level of the structure" while the SEAOC Committee reduces the LEDRS to the "working allowable force level." Thus, R_w should be somewhat higher than R. (For reinforced concrete, $R_w \approx 1.4R$.) In general it can be stated that R_w = (LOAD FACTOR) × R.

It has been very difficult to judge the rationale for and reliability of the values recommended for these R and R_w factors due to a lack of discussion or even any indication of how these values have been derived and what they are meant physically to represent. In reference 9, Chapter 4 of the *Commentary*, it is stated that R "is an empirical response reduction factor intended to account for both damping and the ductility inherent in the structural system at displacements great enough to surpass initial yield and approach the ultimate load displacement of the structural system." In evaluating this statement, it should be noted that the LEDRS selected by ATC is already based on a 5% damped LERS. Therefore, the equivalent viscous damping expected in clean structures should not be significantly greater, particularly in the case of steel structural systems.

Possibly a better explanation of R is given in Chapter 3 of the same *Commentary* [9]. "The response modification factor, R, and . . . have been established considering that structures generally have additional overstrength capacity, above that whereby the design loads cause significant yield." The author believes that this overstrength together with built-in toughness is a "blessing" because of which, or for the primary reasons given above, structures that are designed on the basis of presently specified design seismic forces (UBC or the recommended ATC values) are able or would be able to withstand MCEQ shaking safely. Properly designed (sized) and detailed building structures, when properly constructed and maintained, result not only in considerably higher first "significant effective yielding" than that on which the code design is based, but also offer a significant overstrength beyond the first, effective yielding of the structure. The resulting total overstrength is usually 2 to 3 times greater than the minimum code-specified effective yield strength. This is something that the author has observed since starting to design structures and conduct analytical and experimental research, and fortunately there is now proof of this observation.

The importance of some of the statements included in Section 3-1 of the *Commentary* of the ATC *Recommendations* [9] should be emphasized. "**The values of R must be chosen and used with judgment.** For example, lower values must be used for structures possessing **a low degree of redundancy** wherein all the plastic hinges required for the formation of a mechanism may be formed essentially simultaneously and at a force level close to the specified design strength. This situation can result in considerably more detrimental $P-\Delta$ effects." The importance of the above statement will be illustrated and discussed together with the results of recent research as illustrated in Figures 8-15. However, the implications of these results can be summarized in the statement that **if the building as constructed has a real ultimate strength just equal to the code minimum specified effective yield strength, its response to MCEQ shaking will not be desirable (acceptable).**

Before discussing in detail the implications of recent research results with regard to proposed values for the **modification response factor R and the structural quality factor R_w, it is convenient to discuss the role of ductility in earthquake-resistant design, the reliability of code expressions for estimates of the fundamental period of the structure, T, and the distribution of V throughout the structure.**

3.6.1 Need for Ductility and Its Proper Use in Earthquake-Resistant Design. From analysis of experimental results (both field and laboratory) and analytical studies that the author has carried out, the following observations are made. In earthquake-resistant design, all structural members and their connections and supports should be designed (sized and detailed) with large ductility and stable hysteretic behavior so that the entire structure will also be ductile and display stable hysteretic behavior. There are two main reasons for this requirement: first, it allows the structure as a whole to develop its maximum potential strength which is given by the summation of the maximum strength of each component; and secondly, large structural ductility allows the structure to move as a mechanism under its maximum potential strength and this will result in large dissipation of energy.

While the above two reasons have been recognized in the past, only the second was emphasized because the large dissipation of energy was used to justify the reduction of the design strength that would be required if only linear elastic behavior were permitted. Although this reduction is justifiable in certain cases, the author has expressed, in several previous publications, his concern about **too large reductions** of the required elastic strength (LEDRS) through the indiscriminate use of large values for the structural ductility ratio.

There is no question about the advantage of providing to structural components and their connections (and therefore to the structure as a whole) the

largest ductility that is feasible economically. However, the main reason for doing so should be to provide the structure the opportunity to develop its maximum potential strength according to the maximum strength of its components. The need for this is illustrated in Figure 6 where the strengths of a simple structure composed of a ductile moment-resisting frame and two coupled walls are depicted as the sum of the resistance functions of each of their components. This figure illustrates not only the need for ductility of walls W_1 and W_2, but also the difference between ductility, ductility ratio (μ), and deformability; while the ductile moment-resisting frame has a larger deformability than the walls, its ductility ratio can be smaller than that of the individual walls and this frame ductility ratio cannot be used effectively because of its significantly larger deformability (flexibility) than the wall components, resulting in a relatively earlier failure of the wall components. A quantitative example of this will be illustrated later when the results of the U.S.-Japan Cooperative Research Program are discussed.

While the reduction of the required linear elastic strength R_e obtained by dividing this value by the displacement ductility ratio or ductility factor μ can be justified in the case of structures with a relatively very long period with respect to the period of the predominant frequency content of the earthquake ground motion, the reduction that results by dividing R_e by $\sqrt{2\mu - 1}$ is highly questionable. This reduction can only be justified if the structure is subjected to relatively very short acceleration pulses (with respect to its fundamental period) and the input energy for the linear elastic structure is the same as that for the inelastic (perfectly-plastic) structure. Unfortunately, these assumptions are not realistic in most cases of building response to earthquakes.

It appears that as a consequence of the observed performance of buildings during the 1985 earthquake, Mexican engineers have recognized the previous abuse of the $\mu \equiv Q$ in reducing the LEDRS to obtain C_s. While the 1976 Seismic Code for the Federal District of Mexico allowed engineers to reduce the LEDRS through the use of $\mu \equiv Q = 6$ for the case of ductile moment-resisting space frames (DMRSF) (see Figure 7), the new emergency code, developed as a result of the 1985 Mexican earthquake and already in force, not only mandates a 167% increase in LEDRS, but also a reduction in the value of ductility assumed in the reduction factor Q' from 6 to 4, as illustrated in Figure 7, which means that the C_s for DMRSF has been increased 1.67 x 1.5 = 2.5 times.

3.7 RELIABILITY OF CODE EXPRESSIONS FOR DETERMINING FUNDAMENTAL PERIOD OF STRUCTURES

Present codes permit the value of C_s for a building to be designed on the basis of a deterministic value of T to be estimated by two approaches. The simpler approach is to determine T from empirical equations established using the relatively few experimental data obtained from vibrations measured in existing buildings. The second approach is to determine T based on the mechanical characteristics of the structural system (not of the entire building) in the direction being analyzed and using established methods of mechanics. When this last approach is used, ATC 3-06 requires that the structure be assumed to be fixed at the base of the building.

Analysis of recent experimental data from field tests, records obtained during moderate and/or severe earthquake shaking, and experiments conducted in the laboratory has clearly shown that basing T on either of the two code approaches summarized above can be misleading, i.e., these approaches lead to significant errors in estimating T at the moment that an earthquake strikes and therefore in estimating C_s. This is particularly true in the case of reinforced concrete structures. The source of these errors is illustrated by the results presented in Figures 1, 8, and 9, and in Table 1.

If the effect of infill on moment-resisting frames is neglected, lateral stiffness will be significantly underestimated (Figure 1) and therefore T overestimated. The sensitivity of the flexural and shear stiffnesses of reinforced concrete shear walls to the degree of axial force acting on these walls is clearly indicated in Figure 8. Since the axial force acting on the structure depends on the gravity load (actual dead and live loads that act at the moment that an earthquake occurs), as well as on the change in these axial forces during response to the earthquake ground motion, it is clear that C_s cannot be estimated using just one deterministic value of T.

Figure 9 illustrates the variation of T (as well as those of critical damping ratio) with the accumulation of damage (cracking, yielding, crushing, etc.) to a structure during earthquake response. In 1965, del Valle and Prince [14] reported that periods of six buildings (type K) fourteen stories tall were measured just after construction with the results indicated in Table 1 for building types K-1 and K-5. These periods were measured again after a moderate earthquake that occurred in 1964 in which these buildings suffered minor nonstructural damage with no evidence of structural damage. The periods measured showed increases up to nearly 50% in the longitudinal direction and up to 13% in the transverse (Table 1). It was also reported by del Valle and Prince that the calculated periods for these buildings, K, were longer than the experimentally measured values either before or after the earthquake because partitions had been disregarded in the computations.

Because it is very difficult to predict when (i.e., at what building age) a severe earthquake will occur during the service life of the structure and because T is so sensitive to the effects of infills (nonstructural elements) and to the axial force acting on structural members and on the level of damage (cumulative damage)—as clearly demonstrated in Figures 1, 8, and 9 and by Table 1—

C_s should be determined on the basis of the possible band of values for T for the entire building-foundation-soil system and not just on the basis of one deterministic value of T based on a model of the bare structure assumed to have a fixed foundation.

The importance of the above conclusions can be illustrated by what occurred in Mexico City during the 1985 earthquake. Among the buildings that collapsed or suffered serious damage requiring demolition, the larger number had between seven and fifteen stories. If we look just at the empirical equation presently recommended by the UBC for estimating the fundamental period T of ductile moment-resisting frames, i.e. $T = 0.1N$, N being the number of stories, it appears that for a seven-story building, $T = 0.7$ seconds, and the corresponding spectral value of linear elastic response is $S_a \approx 0.30$ (see Figure 3). This value of S_a is very low compared with the maximum value of S_a of 1.00 that corresponds to $T = 2$ seconds. Therefore, if it is assumed that T should really have been 0.7 seconds, that this period remained constant during building response to the earthquake shaking, and that $R_e \approx 0.30W$, such buildings should not have failed. However, inspection of the buildings of seven or more stories that collapsed revealed that most of them were very flexible (with floor systems based on flat plates or waffle slabs). Thus, the use of $T = 0.1N$ is not realistic for such systems. Furthermore, and of even greater importance, after a few reversals of inelastic deformations: (1) the stiffness of the connections between the flat plate (or waffle slab) and the columns degraded significantly; and (2) the foundations of these buildings (which were not fixed) moved (rocked and slid). Therefore T could increase significantly to values close to 2.0 seconds, explaining the observed damage according to the response spectra shown in Figures 3 and 4.

The experimental results illustrated in Figure 8 are very important for improving the mathematical modeling of reinforced concrete shear walls, particularly in the case of coupled walls where these walls undergo significant changes in axial force during earthquake ground shaking.

Code design procedures and most computer programs for linear analysis of structural systems with identical coupled walls assume that the lateral stiffness of these walls is the same and remains constant during so-called linear elastic response. For two similar walls that are coupled, each wall is assumed to resist, and is therefore designed for, half of the total shear resisted by the coupled walls. The later stiffness (flexural and shear) of reinforced concrete structural elements (particularly walls) is sensitive to the amount of axial force. Therefore, the stiffness of the two coupled walls and consequently the amount of shear resisted by each cannot be the same since, as a result of the coupling girders, the axial force acting in each coupled wall will begin to differ as soon as a lateral force is induced. Therefore, the difference in shear resisted by each wall must increase as the lateral force increases. This has clearly been proven experimentally (see Figure 10).

Existing as well as newly constructed buildings must be carefully instrumented to enable records to be obtained during any ground motion and so to enable an accurate estimation of T. Meanwhile, it will be necessary to conduct forced-vibration tests on existing buildings and buildings under construction to obtain as quickly as possible sufficient and reliable statistical data in order to derive more reliable empirical expressions by which the band of values for T can vary during the service life of various types of structure.

3.8 RELIABILITY OF CODE DISTRIBUTION OF V THROUGHOUT STRUCTURE

3.8.1 Distribution of V Over Height of Structure. For regularly shaped structures or framing systems, the UBC and 1985 SEAOC recommend that V be distributed over height according to equations (2) and (3). For buildings with $T < 0.7$ seconds, and with constant w_i and story height, the distribution follows a triangular shape. The UBC and 1985 SEAOC distributions are illustrated in Figure 11a. For irregularly shaped structures or framing systems, the distribution is to be determined by considering the dynamic characteristics of the structure.

As can be seen from Figure 11a, for a given building of period T, a certain number of stories n, and distribution of dead load w_i, the distribution of V remains the same no matter what structural system is used. This does not seem logical since the vibrational mode shapes of a moment-resisting frame differ considerably from those of a wall.

Figure 11b illustrates the analytically predicted variation with time of total shear and of its distribution along the height of the structure shown in Figure 12a when subjected to the Miyagi-Oki earthquake record. At the time of maximum shear, distribution along the height of the building is quite different from that specified by the UBC and illustrated in Figure 11a. These analytical distributions have been confirmed experimentally [15 & 16].

The distribution of seismic forces along the height, as given by linear and nonlinear dynamic analyses and earthquake simulator tests of the response of frame-wall and braced frame structural systems, show that at the time of maximum axial-flexural (overturning) strength and maximum base shear strength demands, the distributions are quite different. A significant result of the experiments conducted on the structure illustrated in Figure 12a regarding the distribution of total shear along the height of the structure as well as its distribution throughout its components is illustrated in Figure 11c. The present UBC distribution of total V throughout the structure together with current UBC minimum specified shear strength demands for the design of walls against shear is far from conservative. This holds true for the 1985 SEAOC recommendations. While the UBC-specified distribution of V, illustrated in Figure 11a, might be conservative for design against the effect of overturning moment, i.e. flexural design of walls, it is not conservative for design against shear. For buildings with uniform distribution of floor reactive masses along their height and constant story height, it would be better to consider a uniform (rectangular) distribution of total shear (that the structure can resist) rather than the linear distribution suggested by the UBC and 1985 SEAOC and illustrated in Figure 11a.

3.8.2 Distribution of Story Shear in Horizontal Plane. The UBC and 1985 SEAOC recommend that the total shear in any horizontal plane be distributed to the various elements of the lateral force-resisting system in proportion to their rigidities considering the rigidity of the horizontal bracing system or diaphragm. Due to difficulties in predicting the actual stiffness of the floor system in its own plane, reinforced concrete floor systems are usually assumed to behave as infinitely stiff diaphragms. Test results have demonstrated, however, that this is not so for frame-wall lateral force-resisting systems [2]. Since the flexibility of floor systems can lead to significantly different distributions of shear among the different structural elements in a story, the designer should analyze thoroughly the consequences of the possible flexibility of the diaphragm.

4. IMPLICATIONS OF RECENT RESEARCH RESULTS REGARDING RATIONALITY AND RELIABILITY OF VALUES ASSIGNED TO MODIFICATION RESPONSE FACTOR

4.1 INTRODUCTORY REMARKS

As mentioned previously, attempts have been made by ATC [9] to justify the values recommended or proposed for R (or R_w) by implying that this factor is a measure of, or is intended to account for, the ductility inherent in structural systems. If this is the primary rationale used to assign values to R, it is very difficult to understand why for a particular structural system the value of R (or R_w) is a constant for the whole range in which the value of the period, T, can vary for this structural system, as has been recommended or proposed. The studies reported in references 12 and 17 to 20 clearly show that to obtain an inelastic design spectrum from a linear elastic design spectrum, the reduction (de-amplification or modification) factor is a function of the ductility, damping, and characteristics of the resistance function. For any selected resistance function, damping ratio, and ductility, the reduction factor varies with the period of the structure, decreasing as T decreases. For T less than 0.5 seconds, the reduction factor can be 1/2 or even 1/4 of the reduction factor for T greater than 4 seconds, depending on the value of ductility that actually is developed. The greater the ductility, the greater the difference between the reduction factors for structures with long periods as opposed to structures with low (short) periods.

From the above discussion, it appears that the recommendation of a constant value for R (or R_w), i.e., that the value be independent of T for the structure, cannot be justified solely on the basis of the ductility built up in the structure. The values recommended for R (or R_w) appear too high, particularly for short period structures (say, T less than 0.5 seconds) if the designer attempts to provide the structure with only the strength required by the code. Fortunately, as shown in previous publications [1-6] and mentioned earlier in this report, the resulting code design generally produces a significant overstrength. The implications of this observed overstrength are summarized below.

4.2 TEST RESULTS FROM U.S.-JAPAN SEVEN-STORY REINFORCED CONCRETE FRAME-WALL TEST STRUCTURE

4.2.1 Summary. Detailed descriptions and discussions of the design, fabrication, instrumentation, tests, and test results from the experiments and associated analytical studies conducted on this test structure have been published in a series of reports cited in reference 15. Also, the studies conducted at the University of California, Berkeley, are referred to and summarized in reference 19.

Figure 12b, which shows the maximum base shear and base overturning moment resisted by the total (entire) structure and by the wall alone versus the maximum roof drift index, illustrates and summarizes the overall behavior of this test structure. The base shear capacity of the entire structure was 3.75 times the 1979 UBC design demand ($1.4E$) and exceeded that recommended by ATC 3-06 by an even greater margin. The total base shear was resisted by the main wall and ten frame columns. The contribution of the wall to the total shear resistance was 80% during the MO 9.7 test, decreasing to 60% during the T 40.3 test, after which most of the vertical reinforcement of the wall had fractured at its base and the wall was repaired. During this T 40.3 test, only a few of the columns and beams of the frame showed signs of yielding, i.e. at the moment that the wall failed in flexure at its base the DMRSF could not develop maximum yielding strength due to its flexibility, or, in other words, due to the greater deformability of the DMRSF with respect to that of the main wall, it was not possible to take advantage of the displacement ductility supplied to the DMRSF. The flexural resistance of the wall contributed 56% to the total overturning resistance during the MO 9.7 test, the contribution then decreasing to 22% during the T 40.3 test (Figure 12). In this sense the DMRSF system possessed adequate stiffness and strength to compensate for the gradual reduction in the contribution of the main wall. Although the behavior of the test structure was excellent, a somewhat larger supply of stiffness to the DMRSF would have improved overall response, permitting full advantage of the ductility supplied to the DMRSF to be taken.

To facilitate discussion of the implications of these results on values of R, results presented in previous publications [4-6, 15-16] have been replotted in Figures 13 through 15 in the form of pseudo-acceleration spectra.

4.2.2 Actual Period of Test Structure. It can be seen from Figures 13 through 15 that whereas the structure was designed for T = 0.48 seconds, measured T varied significantly during the life of the test structure (Figure 9). Initially, T was measured to be 0.43 seconds, and after a series of test at the service limit states, increased to about 0.61 seconds (i.e. T increased by nearly 50% due to the effect of cracking under service loads). After a series of tests in the damageability limit states, T was about 0.90 seconds just prior to the final test; after this final test, T was 1.16 seconds.

4.2.3 Shaking Table Motion. The shaking table input motion in the test to failure was the Taft earthquake ground motion normalized to 0.40g and with some modification. The 5% damped linear elastic pseudo-acceleration response spectrum (LERS) of the shaking table output is shown in Figure 13. The effective peak acceleration (EPA) does not appear to be defined in the range 0.1 to 0.5 seconds sufficiently well to obtain a reliable value according to the procedure suggested by the ATC [9]. For values of T around the initial value of T of the test structure (i.e. about 0.5 seconds) the EPA seems higher than the 0.40g that would have resulted had the ATC procedure been used.

4.2.4 Implications of Test Results. If the structure had responded only in the linear elastic range, it would have been necessary to design it for a lateral seismic design force coefficient C_s greater than 1.00 (Figure 13). The structure was designed according to the UBC, and the UBC required-minimum yield strength is equivalent to $(C_s)_y = 0.11$. However, considering that the UBC requires the wall alone to resist all the code-specified lateral force and the ductile moment-resisting space frame (DMRSF) to resist at least 25% of the required lateral force, the combined minimum required design strength is equivalent to $(C_s)_y = 0.14$. This corresponds to a reduction of the 5% damped LERS of the shaking table output of more than 8 (see Figure 13) and of about 7 with respect to the 5% damped LEDRS recommended by ATC (see Figure 14).

The experimental results show that the first significant yielding of the wall occurred under a $(C_s)_y \approx 0.18$, but that the maximum strength was $(C_s)_{max} \approx 0.51$ (Figures 12-13). This confirms the earlier conclusion that for a well-designed structure, effective yielding occurs after a significantly higher value than the minimum code-required value ($0.18/0.11 = 1.64$ or $0.18/0.14 = 1.29$) has been attained, and that the actual maximum strength shows a considerable overstrength beyond the first effective yielding ($0.51/0.18 = 2.83$). This is really a blessing because it allows UBC-designed structures to withstand safely the effects of the maximum credible earthquake (MCEQ) shaking. If the structure as designed had had as its actual maximum strength the minimum required by the UBC (i.e. $(C_s)_{max} = 0.14$), its performance under the Taft 0.40g motion would probably not have been acceptable. To dissipate the input energy from the Taft 0.40g motion, it would have been necessary for a structure with $(C_s)_{max} = 0.14$, to have an interstory drift ratio significantly higher than the maximum measured during the tests. This maximum was 1.7% [5-6, 15-16], already greater than the maximum of 1.5% specified by the ATC.

Figure 14 illustrates the relationship between the ATC-recommended lateral design force coefficients C_s and the 5% damped LERS from which they have been derived when applied to the seven-story reinforced concrete test structure for a site in a seismic region similar to that of San Francisco. If the structure had been designed according to the C_s recommended by ATC, its $(C_s)_y$ would clearly have been less than 0.125. In fact:

$$(C_s)_y = \frac{1.2 \times 0.40 \times 1.0}{8 \times 0.48^{2/3}} \approx 0.11$$

which is significantly lower than the UBC-required combined $(C_s)_y$ of 0.14.

The results plotted in Figure 15 allow the required yielding strength if the structure were to remain linear elastic and damped with ξ = 5% under the actual shaking table output to be compared with: (a) the spectral shape used by ATC for sites when A_a and A_v are 0.40 and the corresponding seismic design coefficients using the recommended R = 8 for the dual system of the test structure; (b) the yielding strength for which the structure was designed, $(C_s)_y$ = 0.14; and (c) the measured strength $(C_s)_{max}$ = 0.51. If the design test structure had been built on rock or stiff soil (soil profile S_1) then the maximum strength of this structure $(C_s)_{max}$ = 0.51 in the final test (where the effective period T of the structure was ≈ 1 second) would have been equal to the yield strength required by the ATC 5% damped LERS. Despite this, the measured interstory drift ratio (1.7%) exceeded the maximum value recommended by ATC (1.5%).

The actual value of R, termed R_a, of the structure would have varied depending on the state of the structure (cumulative damage) at the moment that the table output (simulating Taft 0.40g) occurred (Figure 15). If this ground motion had occurred just after construction, with $T \approx$ 0.50 seconds, then R_a could have been as large as $1.4/0.51 \approx 2.7$. On the other hand, if the shaking had occurred after the structure had already been damaged by moderate earthquake shaking and/or strong wind, so that the value of T would already have increased to about 0.8 seconds, then R_a could have been as low as 1.5, and even lower if damage to the structure had increased T to a value of about 1.0 second.

If instead of the 5% damped LEDRS of the table output, the 5% LEDRS specified by ATC for soil S_3 were considered, R_a is about 1.6 for T up to around 0.9 seconds. The R_a values given above are the effective values for which the 5% LEDRS can be reduced to attain the **actual maximum shear resistance** that the structure has to possess in order to perform well if it is subjected to ground shaking with a 5% damped pseudo-acceleration similar to that specified by ATC or that corresponding to the shaking table output for the 0.40g Taft.

When the above values of R_a are compared with the value of R recommended by ATC (8 for dual systems based on reinforced concrete shear walls and reinforced concrete ductile moment-resisting frames), it is clear that it is not wise to try to provide a structure with just the ATC-required minimum first significant yielding strength. For the type of structure tested, i.e., that illustrated in Figure 12a, it is necessary to ensure that the resulting ATC design has an actual maximum strength equal to R/R_a, i.e., 8/1.6 or approximately 5 times the ATC-minimum required significant yield level. Clearly, therefore, a designer who uses ATC (or UBC) should check any design based on the minimum specified code forces; such designs must have a maximum strength significantly higher than that demanded by the code factors.

To summarize, it can be concluded that it is very difficult to rationalize (justify) quantitatively the values recommended by ATC for R [9]. If the value of R alone is used in the design of reinforced concrete frame-wall dual systems, i.e. **without any other requirements**, the resulting design will not be reliable. The use of a specific value for R should be tied to other requirements. In the present ATC recommendations [9], the value of R is tied to stringent requirements for detailing reinforced concrete ductile moment-resisting space frame members and structural walls (see Appendix A, ACI 318-83 [21]). However, the author believes that this is not enough, and suggests that the preliminary design resulting from the ATC-recommended approach (or that of the UBC) be subjected to a limit analysis to obtain an estimate of the actual maximum resistance of the structure as designed, and that a value approximately 5 times the minimum yielding strength required by ATC be ensured for structural types such as that illustrated in Figure 12a. Furthermore, the design of the wall (sizing and detailing) against shear (as well as against shear of members of ductile moment-resisting space frames) should be based on this maximum resistance.

The SEAOC Seismology Committee [7] has suggested that R_w = 12 for dual systems composed of reinforced concrete frames and shear walls. Studies and comparisons similar to those discussed above for the value of R indicate that this is a little less conservative than the ATC ($1.4/12 < 1/8$), and therefore significantly higher than the R_a value measured experimentally (Figure 15). Therefore, the observations made above for the case of R apply similarly to the case of R_w.

4.3 TEST RESULTS FROM U.S.-JAPAN SIX-STORY STEEL FRAME/BRACED FRAME TEST STRUCTURES

Two dual systems—one based on a concentric braced frame and a special (ductile) moment-resisting space frame (SMRSF) and the other on an eccentric braced frame and SMRSF—were tested in Tsukuba, Japan and in Berkeley, California. Results of this study were summarized in reference 3 and discussed in more detail in references 22 and 23, in which comparisons similar to those made above for the reinforced concrete frame wall test structure were described. These comparisons resulted in observations similar to those made for the case of the reinforced concrete test structure and also led to the conclusion that for these types of steel frame dual systems, values for R and R_w cannot be as large as is presently recommended by ATC and the SEAOC Seismology Committee.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

In what follows, some general observations and/or conclusions, particularly regarding the adequacy of present U.S. seismic-resistant design regulations and recently recommended changes in these regulations, are formulated. The results illustrated in Figure 16 are first discussed in order to emphasize the importance of the data and results from the 1985 Chilean and Mexican earthquakes and the implications of these for U.S. seismic regulations and particularly for changes suggested by SEAOC in 1985.

It is clear from Figure 16a that the LEDRS assumed by SEAOC and therefore ATC are significantly smaller than those that occurred in the 1985 earthquakes in Mexico and in Chile. If such ground motions were to occur in the U.S., the actual value of the reduction factor (R or R_w) could well be twice that specified in the code, if the maximum yielding strength of structures as constructed equaled the minimum required by the Code and represented in Figure 16 by $(V/W)_n$.

Figure 16b illustrates the significant difference in the earthquake-resistant design regulations mandated by the 1985 Mexican emergency code and those specified by the 1985 SEAOC recommendations for buildings of special occupancy (SEAOC category III and Mexico group B) built on soft soil (SEAOC S_3 type and Mexico zone III) and with a reinforced concrete (RC) DMRSF. The required nominal yielding strength $(V/W)_n$ of the 1985 Mexican code is higher in general, and significantly higher (by more than 2 times) for period T around 3 seconds (Figure 16b). The significant difference between the basic LEDRS and the reduction factors (R_w vs Q') used to obtain the seismic design forces are also clearly illustrated. Hospitals and schools are included in group A in the 1985 Mexico Code while they are considered to be of occupancy category III in the 1985

SEAOC tentative requirements. Thus, the $(V/W)_n$ required for these buildings by the Mexico Code will be more than 1.5 to 3 times that required by SEAOC.

1. The values specified in seismic codes for C_s depend, among other factors, on three main parameters: (1) the LEDRS for the seismic zone on which the building is or will be constructed; (2) the reduction factor R (or R_w) for the LEDRS to obtain what can be considered the IDRS; and (3) the estimation of the fundamental period, T, of the building.

2. The statistical information available for the determination of the above three parameters has been scant and any additional information that has become available from: (1) records of any new moderate and/or major earthquake ground shaking and the processing and analyses of these records; (2) the performance of structures during such ground motions; and (3) the results of experimental and associated analytical studies have dramatically changed the previous statistical bases.

3. The ground motions recorded in zone III of Mexico City during the 19 September 1985 earthquake significantly exceeded the intensity (by more than three times) and the duration of strong motion previously anticipated and considered in the 1976 seismic code for the Mexico DF. This ground motion also significantly exceeds that expected in the zones of highest seismic risk in the U.S. for similar soil conditions.

4. The 5% damped LERS for the recorded E-W component of the ground motion at station SCT of Mexico City (zone III) for T between 1.7 and 3 seconds exceeded those considered by ATC and SEAOC (1985) for soft soils. For $T \approx 2$ seconds, the spectral coordinate of the SCT's LERS is more than twice that assumed by ATC and SEAOC.

5. The recorded N10E component of the ground motion at LLolleo during the 3 March 1985 Chilean earthquake has a PGA of 0.67g and an EPA significantly

greater than the maximum recommended by ATC in the U.S. (0.40g). The 5% damped LERS for T between 0.1 seconds to nearly 2 seconds of the LLolleo record significantly exceeds the similarly damped LEDRS considered by ATC and SEAOC (1985) for firm soils in the regions of highest seismic risk in the U.S. For T of about 0.2 and 0.6 seconds the spectral coordinates of the N10E LLolleo 5% LERS are more than 2.1 times those assumed by ATC and SEAOC.

6. When the LERS corresponding to the recorded ground motions at SCT (Mexico City) and LLolleo (Chile) are compared with the intensity of these ground motions, the resulting amplification factors are considerably higher than the values presently used (and/or recommended) in the U.S., even for a probability level of One Sigma.

7. The recorded motions at SCT and LLolleo show long durations of strong motion, longer than any previously recorded, indicating that code-designed structures can undergo a significant number of reversals with high ratios of ductility demand, considerably greater than was considered possible before these 1985 earthquakes.

8. Comparisons of the recorded ground motions and the observed performance of building structures, particularly in Mexico City, clearly demonstrate the importance of soil-building resonance and the need for designers carefully to consider the phenomenon. It is therefore extremely difficult to justify the proposal of the SEAOC Seismology Committee that T/T_s need no longer be computed.

9. Many buildings in Mexico City suffered serious damage due to hammering of adjacent buildings against each other due to a lack of proper separation. Although present U.S. seismic regulations regarding building separation are significantly less stringent than those of the apparently inadequate requirements of the Mexican Codes, the large separation that would be required to

avoid such hammering of adjacent high-rise buildings realistically cannot be economically realized, especially with respect to existing structures. It will therefore be necessary to develop other and compromise regulations or requirements to avoid the degree of damage from hammering seen in Mexico City. Economical solutions for retrofitting existing adjacent structures with inadequate separation should be the subject of immediate research.

10. Estimation of a deterministic value of T based on present U.S. Seismic codes approaches can be misleading as to required strength (C_s) and stiffness demands. These demands should be estimated on the basis of the possible band of values through which T can vary according to the uncertainties involved in the estimation of the mechanical characteristics of the building-foundation-soil system. The reliability of the empirical expressions presently used to estimate T urgently needs review.

11. The minimum seismic forces specified by the UBC and those recommended by ATC and the Seismology Committee of SEAOC in their revision of the *Blue Book* are **unrealistically low when compared with the seismic forces that occur in code-designed structures.**

12. The present ATC and SEAOC recommendation that R and R_w be calculated independently of T cannot solely be justified on the basis of ductility built-up in the structure. The recommended values appear to be too high, particularly for short-period structures and when the designer attempts to provide a structure with just the minimum strength required by these codes.

13. Fortunately, in most cases, and particularly for dual structural systems, the resulting code-designed structures have significant axial-flexural over-strengths. Unfortunately, the wall shear overstrength and the axial-flexural strength of braces could be significantly less than this overall axial-flexural overstrength, possibly leading to premature shear and buckling failure. To

avoid this it is necessary that actual overstrengths be estimated as accurately as possible.

14. While present code distributions of V throughout the structure may be conservative for design against the effect of overturning moment, they are neither conservative nor realistic for design against shear.

5.2 RECOMMENDATIONS

Present procedures for the seismic-resistant design of building structures could be improved by the recognition and implementation of two methods described below: a "Rational Method" and a "Compromise Method."

Rational Method. The design should be based on a reliable inelastic design response spectra (IDRS) and should consider the probable actual three-dimensional strength of the whole soil-foundation building system, i.e. the preliminary design should be performed considering safety against collapse as the controlling limit state.

Compromise Method. As at present there are not sufficient reliable data from which to formulate reliable IDRS and to predict the actual three-dimensional supplies of strength to soil-foundation building systems, the following design procedure should be implemented as a compromise until sufficient such data become available.

1. While the preliminary design could be performed according to procedures presently recommended in U.S. seismic codes, i.e. based on LEDRS, improved methods of estimating the weight W of the probable reactive mass and T and C_{sp} (i.e. more reliable values for R and R_w) should be used.

2. Based on this preliminary design, the probable three-dimensional axial-flexural strength of the soil-foundation-building system should be estimated as accurately as possible.

3. Based on the estimate of three-dimensional axial-flexural strength as described in '2.' above, the maximum shear axial strength demands (including buckling and anchorage forces) at the critical regions of the entire structural system should be determined and these critical regions should be designed and detailed to resist such maximum shear, axial (buckling), and anchorage forces.

The author emphasizes his conviction that earthquake resistance cannot be significantly enhanced simply by increasing the seismic forces presently specified in U.S. seismic codes. What is proposed is that the forces developed during earthquake shaking be recognized to depend on the actual stiffness, strength, and hysteretic characteristics supplied to the constructed building. What must be developed is an accurate method of estimating the three-dimensional capacity of the entire soil-foundation-building system, not simply that of the bare superstructure. Although there is an obvious need to improve earthquake-resistant design procedures, there is an even greater need to improve construction and maintenance procedures if the earthquake hazard is to be mitigated.

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*** T A B L E S ***

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TABLE 1 TYPE K BUILDINGS—MEASURED PERIODS (secs) (After [13])

BUILDING	*K*-1		*K*-5	
DIRECTION	LONGITUDINAL	TRANSVERSE	LONGITUDINAL	TRANSVERSE
AFTER COMPLETION	1.04	1.68	1.07	1.54
AFTER EARTHQUAKE	1.54	1.90	1.24	1.72
AFTER REPAIR	1.09	1.70	1.09	1.57

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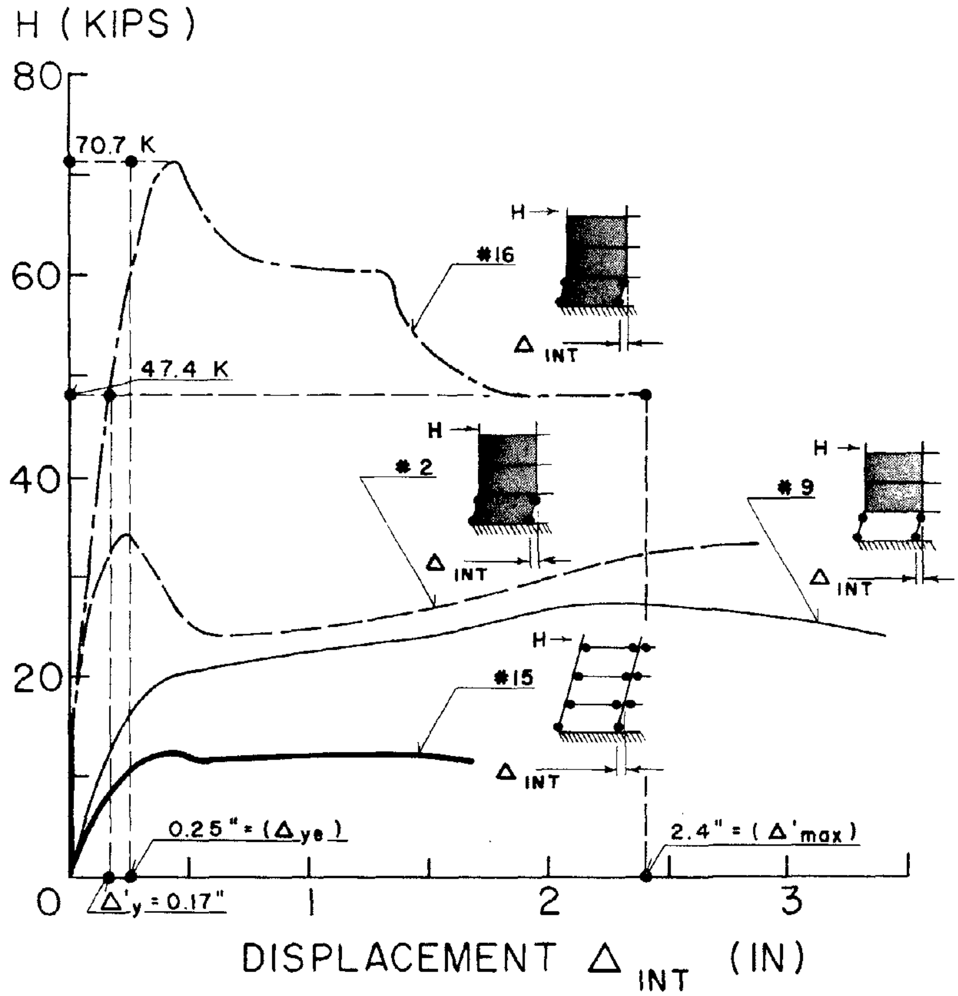
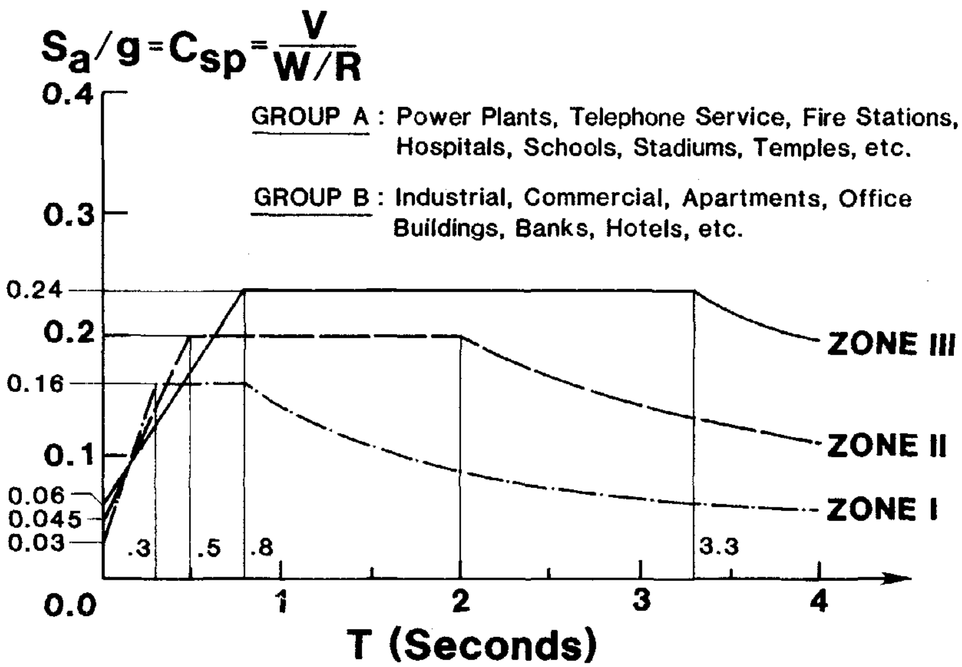
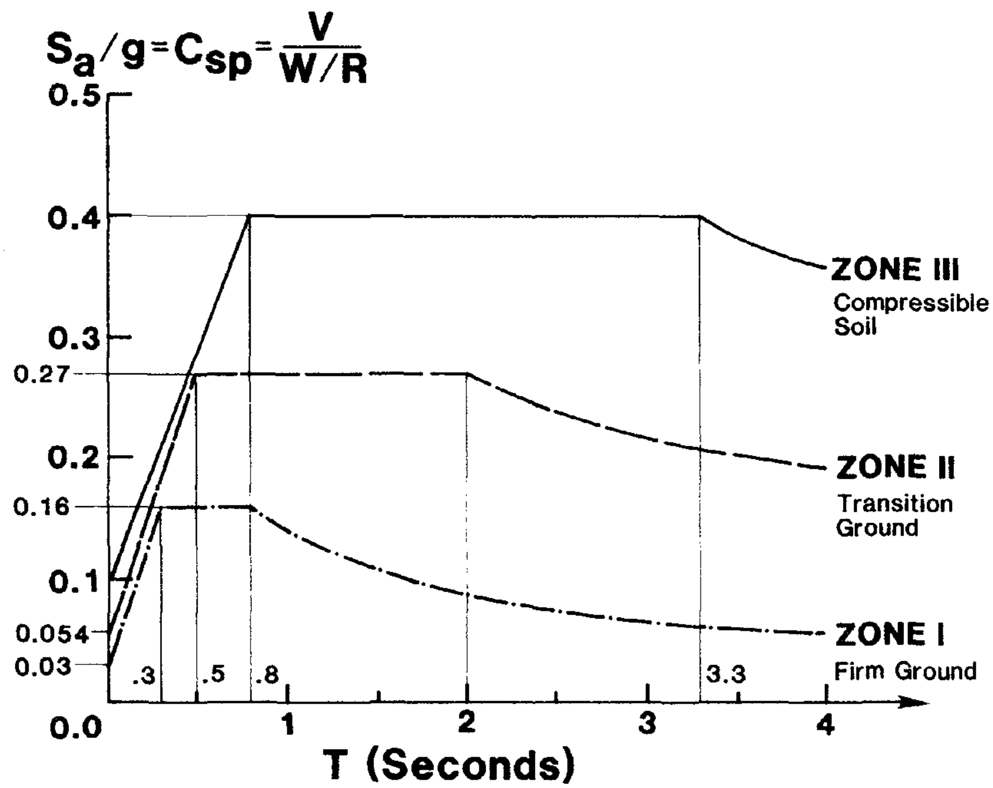


FIGURE 1 EFFECTS OF ADDITION OF INFILLS ON LATERAL LOAD (H) vs INTERSTORY DRIFT (Δ_{INT}) RELATIONSHIP FOR MOMENT-RESISTING FRAMES

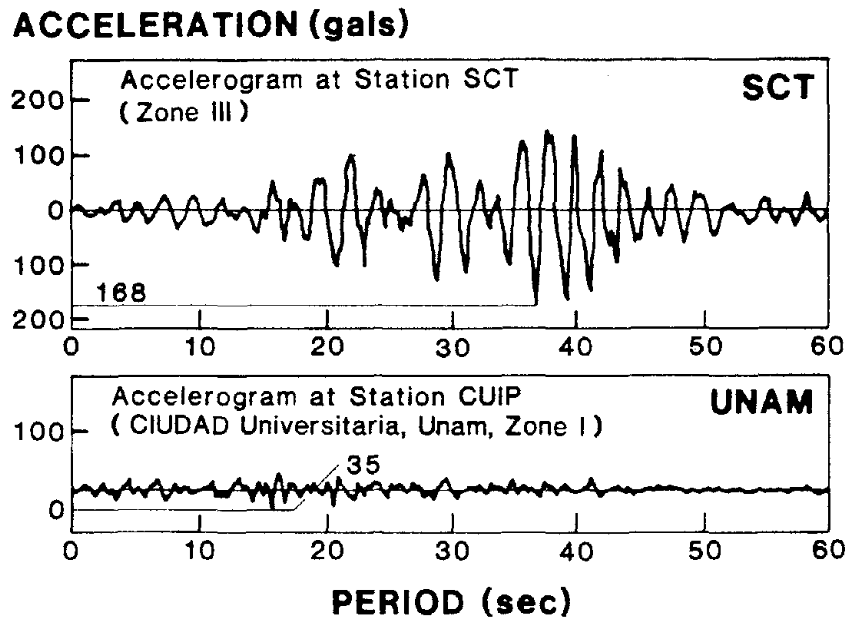


2a 1976 CODE LEDRS FOR GROUP B BUILDINGS;
FOR GROUP A, SPECTRAL VALUES SHOULD BE MULTIPLIED BY 1.3

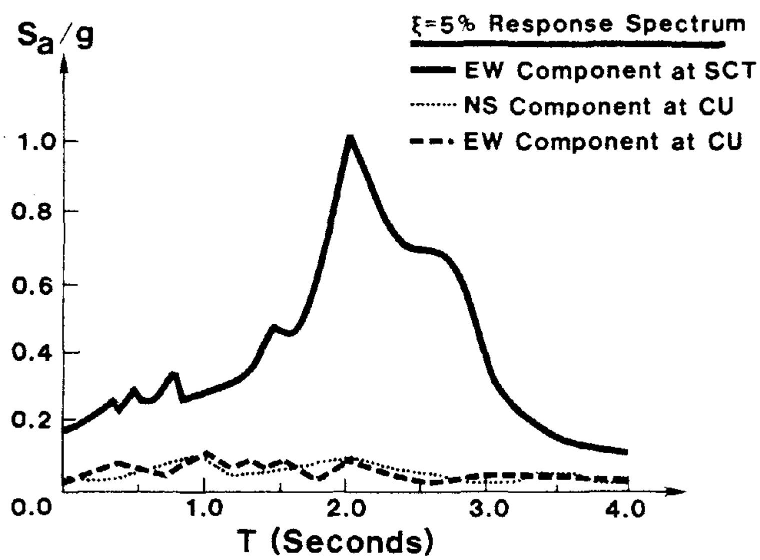


2b 1985 EMERGENCY CODE LEDRS FOR GROUP B BUILDINGS;
FOR GROUP A, SPECTRAL VALUES SHOULD BE MULTIPLIED BY 1.5

FIGURE 2 5% DAMPED LINEAR ELASTIC DESIGN RESPONSE SPECTRA (LEDRS) FOR FEDERAL DISTRICT OF MEXICO



3a RECORDED EW COMPONENT OF GROUND MOTIONS



3b 5% DAMPED LINEAR ELASTIC RESPONSE SPECTRA OF SOME RECORDED COMPONENTS OF GROUND MOTIONS

FIGURE 3 19 SEPTEMBER 1985 MEXICAN EARTHQUAKE: EW COMPONENTS OF GROUND MOTIONS RECORDED AT STATION CU (CIUDAD UNIVERSITARIA) AND AT SCT (SECRETARY OF COMMUNICATION AND TRANSPORTATION) AND 5% DAMPED LINEAR ELASTIC RESPONSE SPECTRA

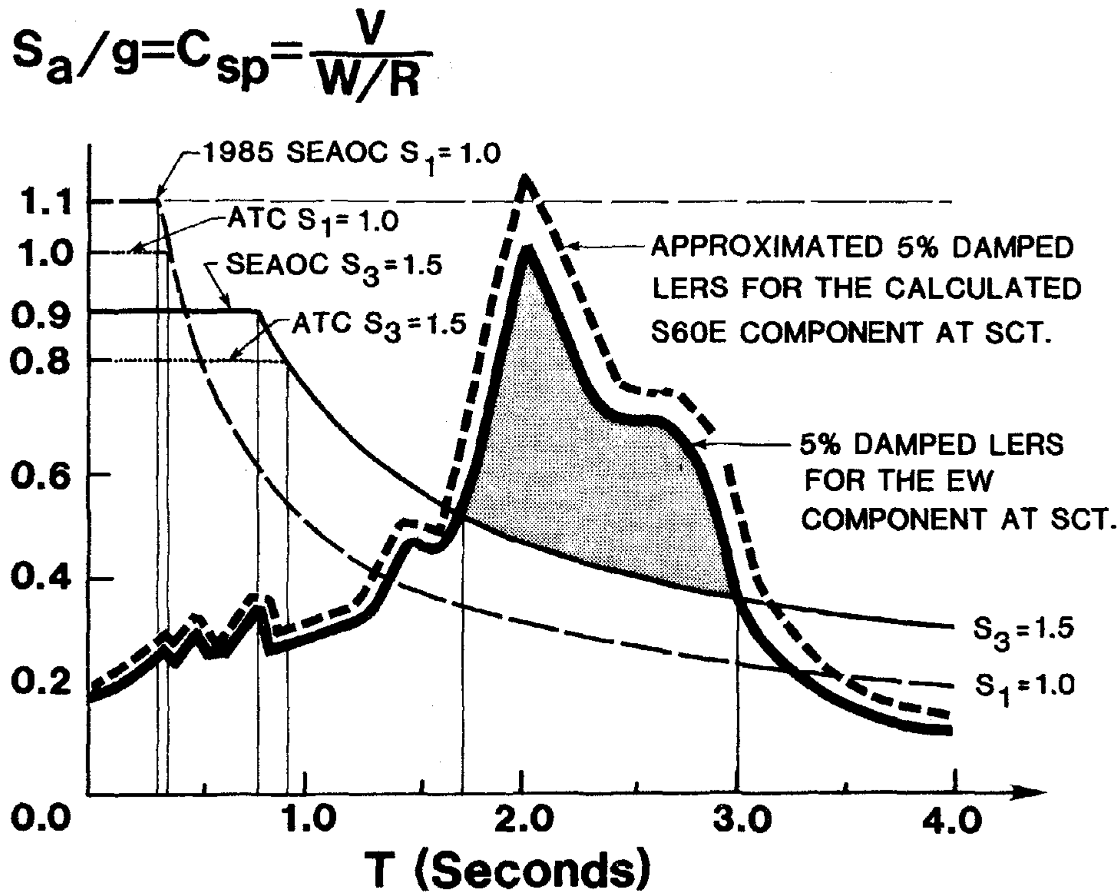


FIGURE 4 COMPARISON OF 5% DAMPED LINEAR ELASTIC RESPONSE SPECTRUM (LERS) FOR EW COMPONENT RECORDED AT STATION SCT DURING 19 SEPTEMBER 1985 MEXICAN EARTHQUAKE WITH 5% DAMPED LINEAR ELASTIC DESIGN RESPONSE SPECTRA (LEDRS) RECOMMENDED BY ATC 3-06 AND SEAOC (1985) FOR REGIONS OF HIGHEST SEISMIC RISK IN U.S.

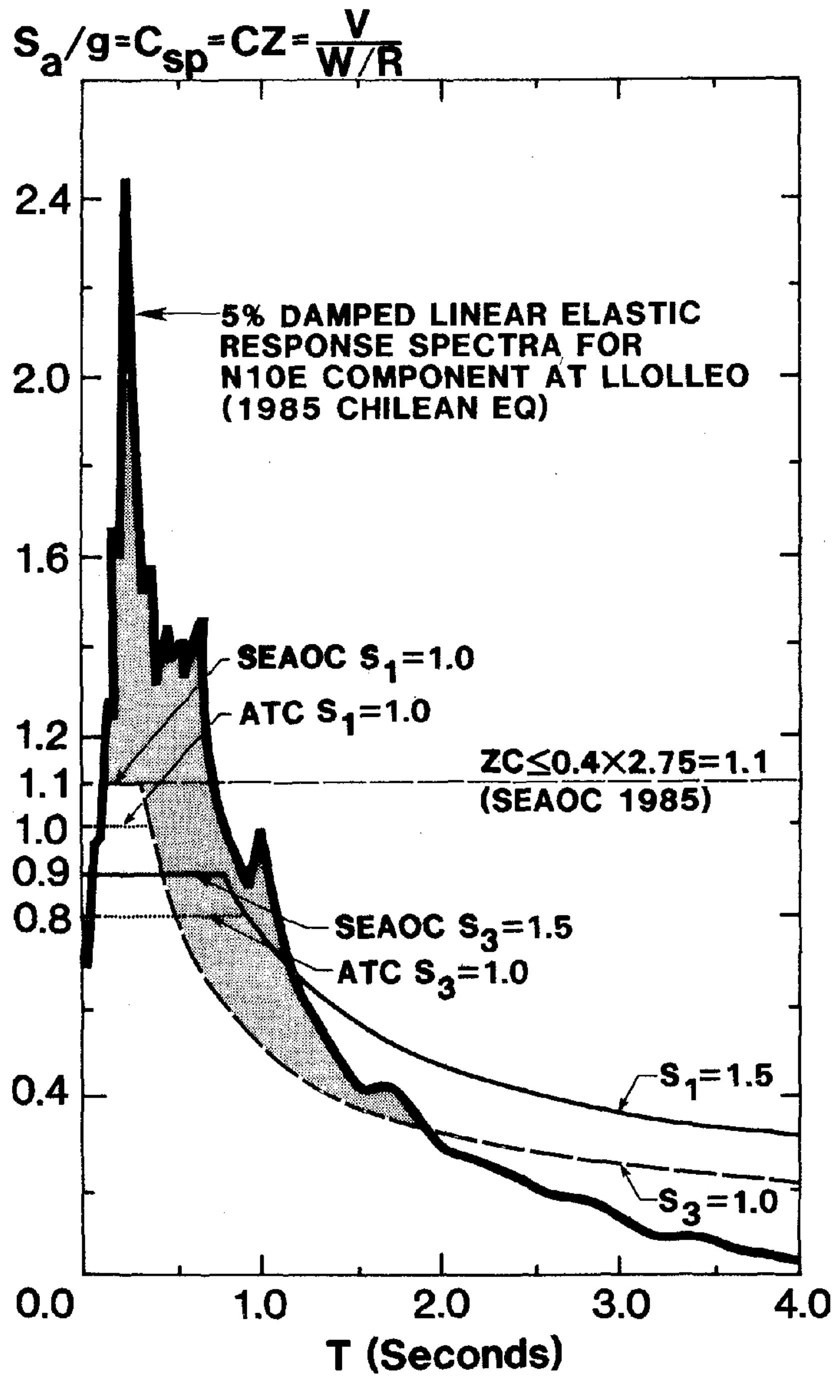


FIGURE 5 COMPARISON OF LINEAR ELASTIC RESPONSE SPECTRUM FOR N10E COMPONENT RECORDED AT LLOLLEO DURING 3 MARCH 1985 CHILEAN EARTHQUAKE WITH 5% DAMPED LEDRS RECOMMENDED BY ATC 3-06 AND SEAOC (1985)

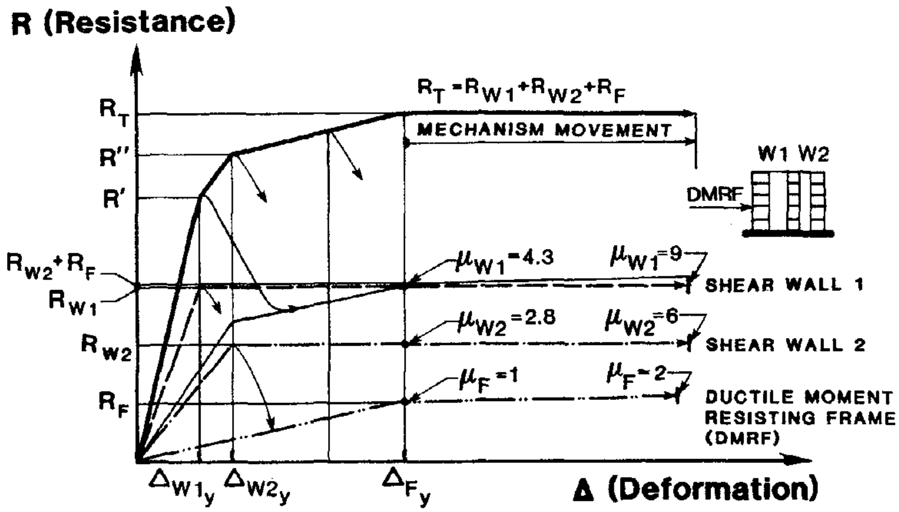


FIGURE 6 ILLUSTRATION OF NEED TO PROVIDE DUCTILITY (μ) TO ALL SUB-STRUCTURAL SYSTEMS (W_1, W_2, DMRF) AND STRUCTURAL COMPONENTS (COLUMNS, GIRDERS, CONNECTIONS, SUPPORTS) TO ALLOW ENTIRE STRUCTURE TO DEVELOP MAXIMUM POTENTIAL STRENGTH (R_T) GIVEN BY SUMMATION OF MAXIMUM STRENGTH OF EACH COMPONENT ($R_T = R_{W_1} + R_{W_2} + R_F$), AND TO ALLOW STRUCTURE TO MOVE AS A MECHANISM UNDER MAXIMUM POTENTIAL STRENGTH.

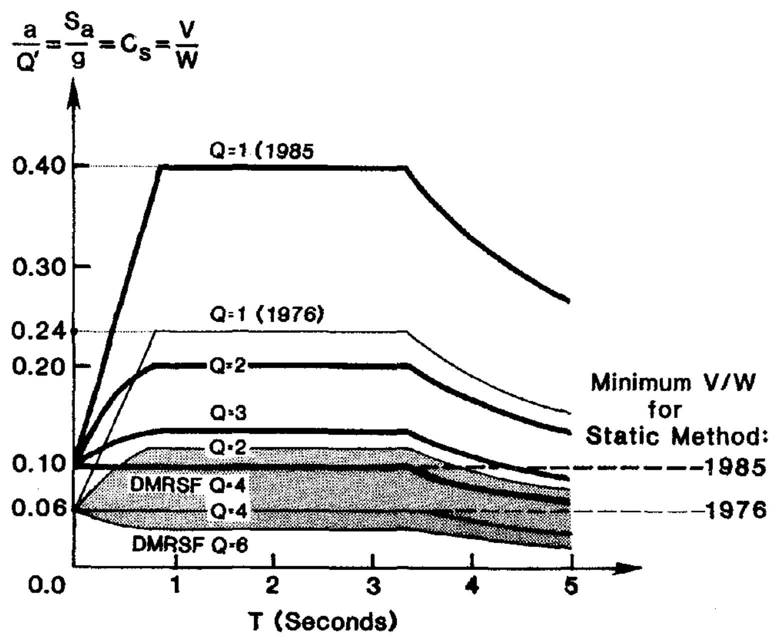
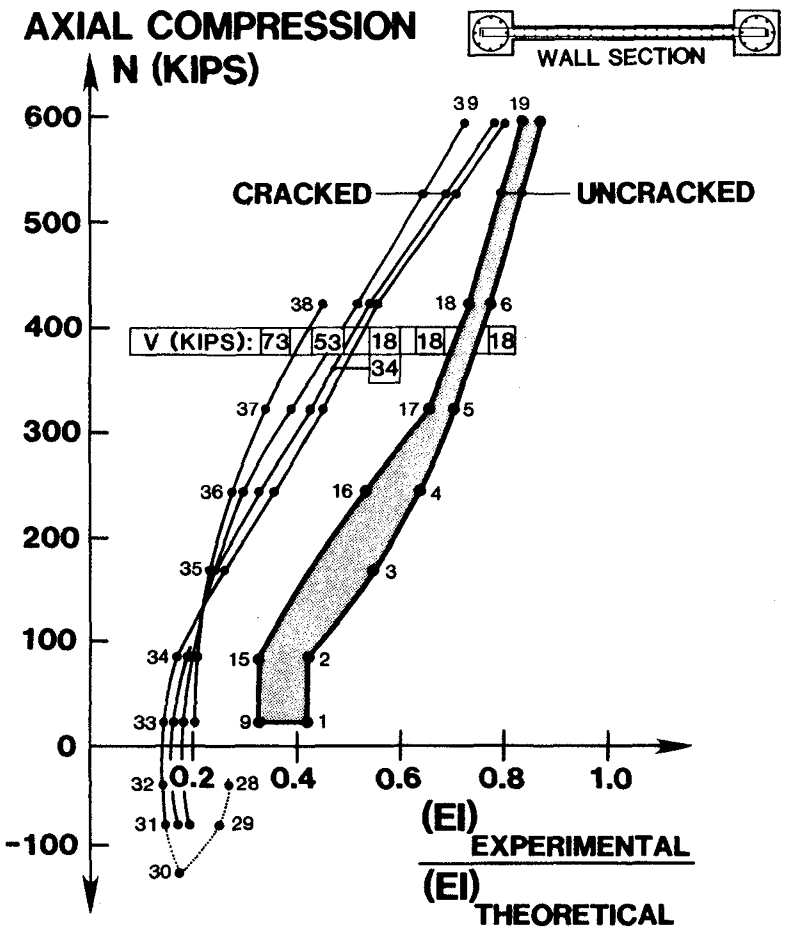
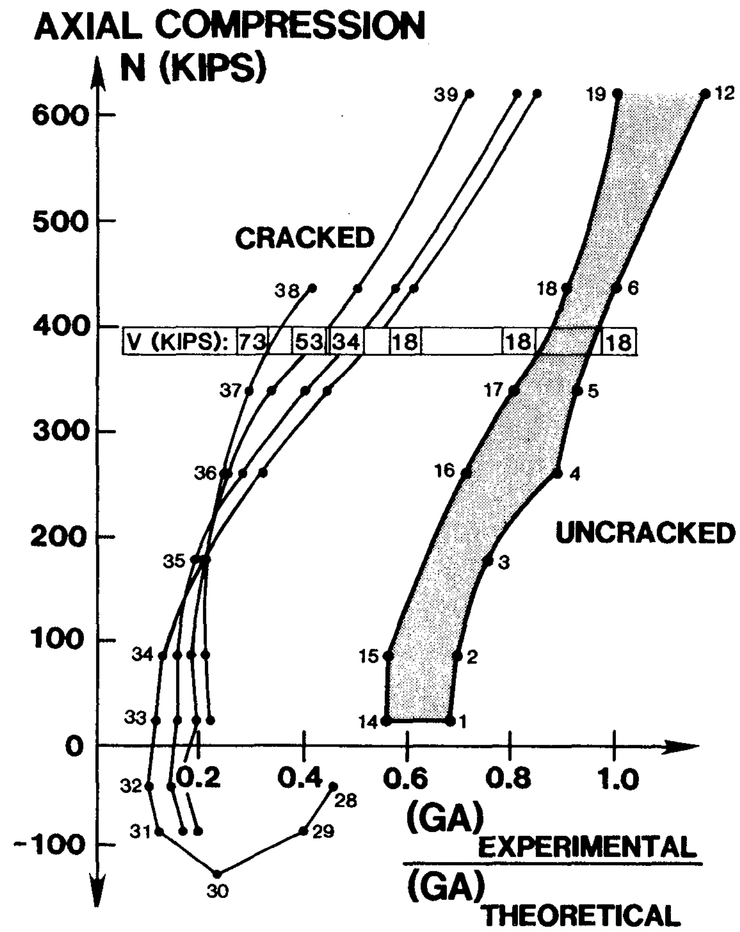


FIGURE 7 COMPARISON OF REDUCED DESIGN SPECTRA (DUE TO DUCTILITY Q) FOR BUILDINGS IN GROUP B LOCATED IN ZONE III OF FEDERAL DISTRICT OF MEXICO: 1976 AND 1985 EMERGENCY CODES



8a VARIATION OF FLEXURAL STIFFNESS (EI)



8b VARIATION OF SHEAR STIFFNESS (GA)

FIGURE 8 REINFORCED CONCRETE SHEAR WALL: VARIATIONS OF FLEXURAL STIFFNESS (EI) AND SHEAR STIFFNESS (GA) WITH MAGNITUDE OF AXIAL AND SHEAR FORCES ($M/V = 0.29H$)

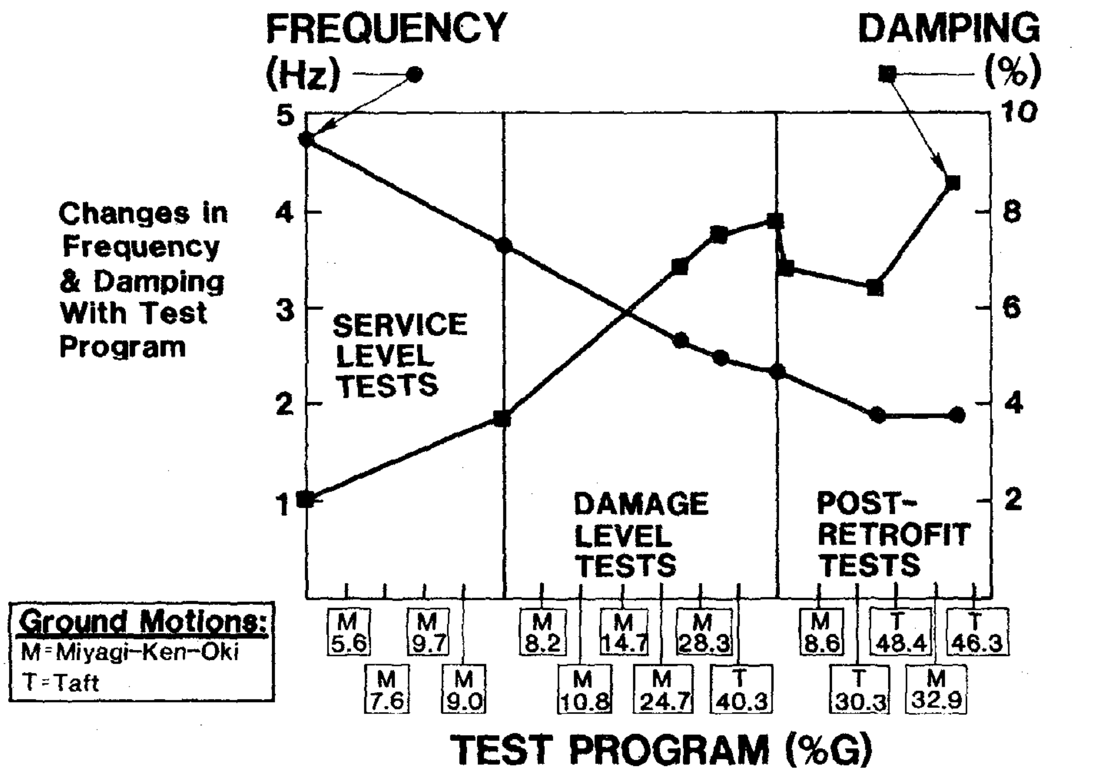


FIGURE 9 VARIATION OF FREQUENCY ($1/T$) AND DAMPING RATIO OF ONE-FIFTH-SCALE MODEL OF SEVEN-STORY REINFORCED CONCRETE FRAME-WALL TEST STRUCTURE [16]

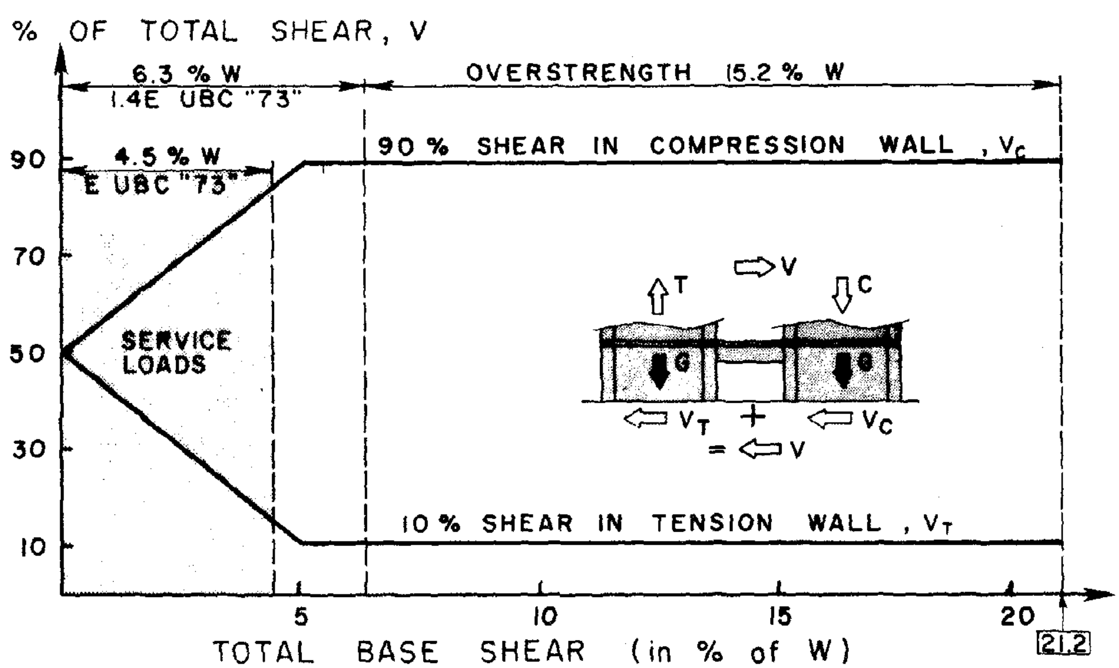
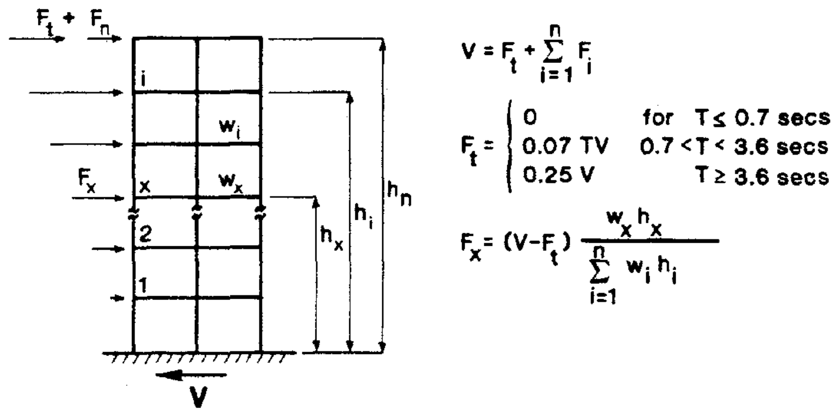
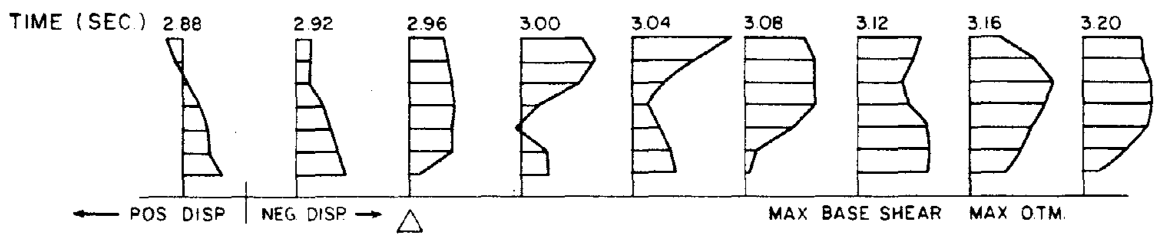


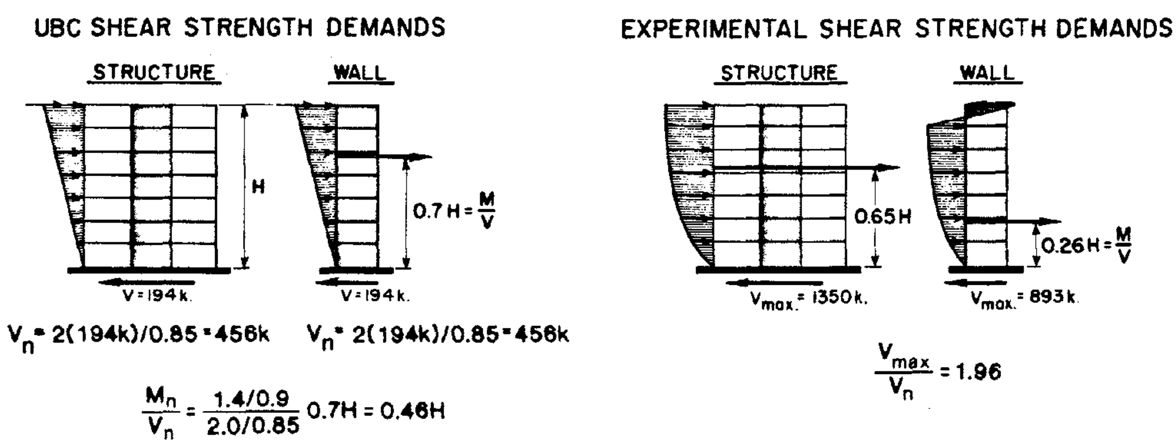
FIGURE 10 MEASURED REDISTRIBUTION OF TOTAL BASE SHEAR OF A COUPLED WALL SUBASSEMBLAGE TESTED AT U.C., BERKELEY [2, 4]



11a UBC 1982 AND 1985 DISTRIBUTIONS OF TOTAL LATERAL SEISMIC FORCE *V* ALONG BUILDING HEIGHT

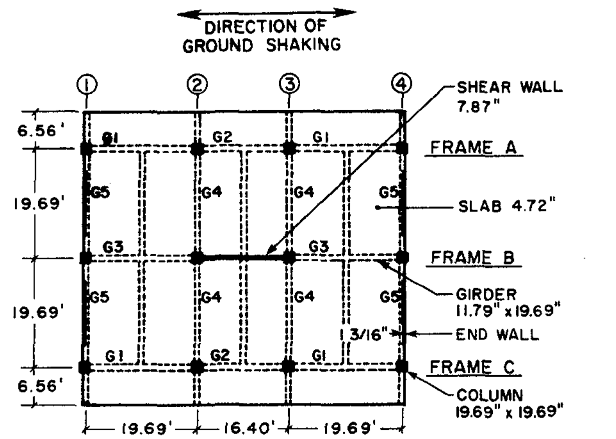


11b LATERAL FORCE PROFILE-TIME HISTORY FROM RESPONSE OF STRUCTURE IN FIGURE 12a TO MIYAGI-OKI GROUND MOTION

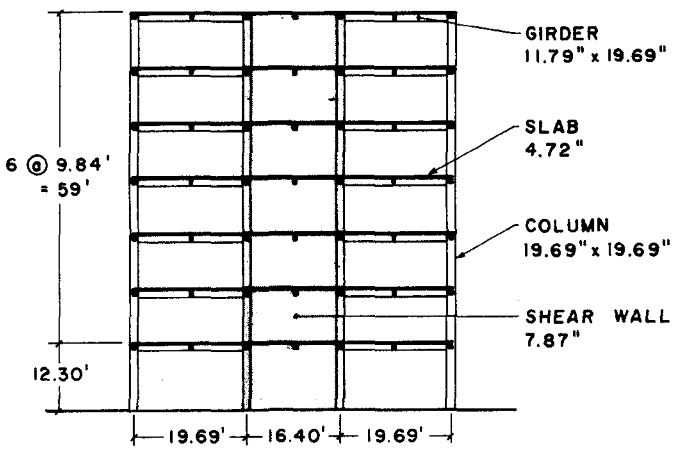


11c EFFECT OF VERTICAL DISTRIBUTION OF *V*: 1979 UBC MINIMUM NOMINAL SHEAR STRENGTH DEMAND (BASED ON UBC RECOMMENDED LINEAR DISTRIBUTION OF *V*) vs EXPERIMENTAL DEMAND VALUES FOR TOTAL STRUCTURE AND WALL IN FIGURE 12a

FIGURE 11 DISTRIBUTION OF TOTAL LATERAL SEISMIC FORCE (*V*) OVER HEIGHT OF STRUCTURE OF FIGURE 12a AND EFFECT ON DEMANDED NOMINAL

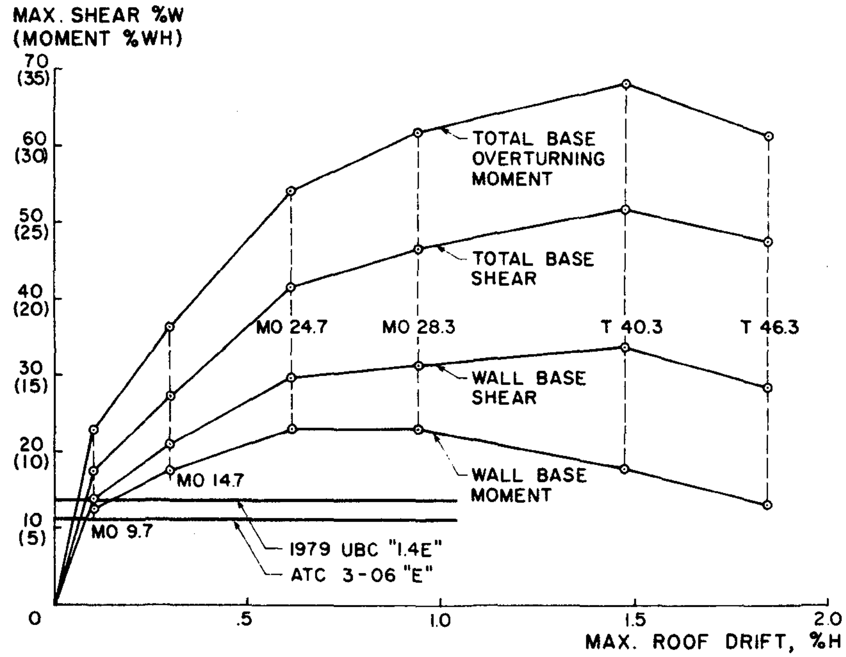


(a) PLAN WITH FRAME AND GIRDER LOCATIONS



(b) SECTION SHOWING FRAME B

12a PLAN AND SECTION



12b ENVELOPES OF MAXIMUM BASE SHEAR AND BASE OVERTURNING MOMENT vs MAXIMUM ROOF DRIFT INDEX OF TOTAL STRUCTURE AND WALL ALONE, AND MINIMUM CODE STRENGTH REQUIREMENTS

FIGURE 12 U.S.-JAPAN REINFORCED CONCRETE STRUCTURE AND COMPARISON OF GLOBAL SEISMIC RESPONSE WITH MINIMUM REQUIRED CODE STRENGTH

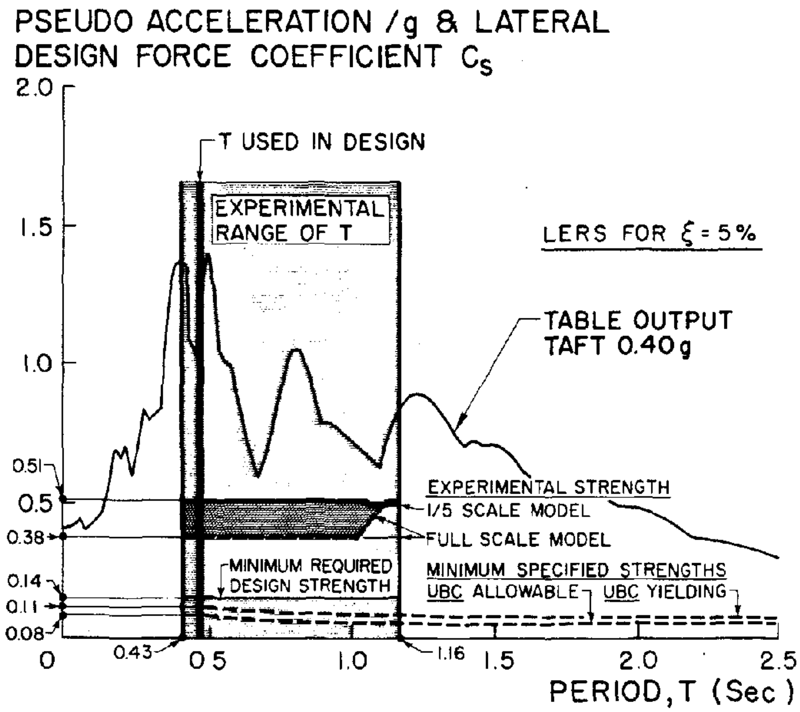


FIGURE 13 COMPARISON OF UBC MINIMUM SPECIFIED LATERAL SEISMIC STRENGTHS WITH REQUIRED STRENGTH FOR A 5% DAMPED LINEAR ELASTIC RESPONSE TO SHAKING TABLE MOTION AND WITH MEASURED STRENGTHS FOR STRUCTURE IN FIGURE 12a

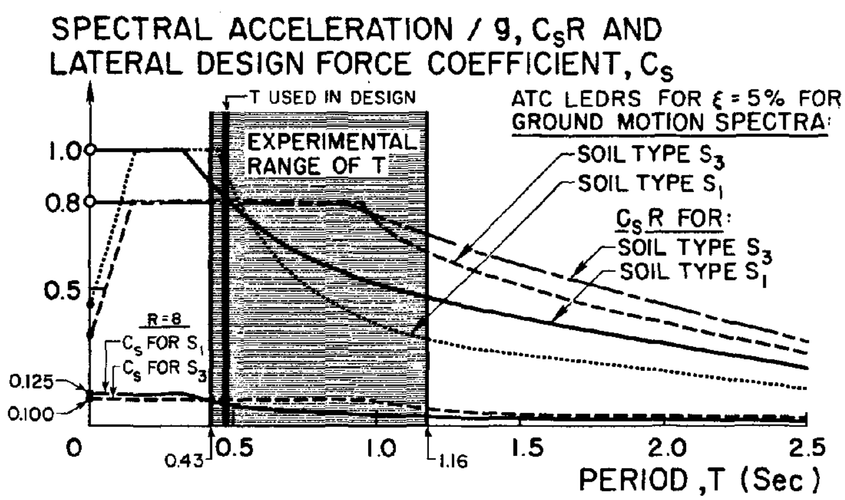


FIGURE 14 ATC 5% DAMPED PSEUDO-ACCELERATION SPECTRA FOR FREE-FIELD GROUND MOTIONS AND LATERAL SEISMIC DESIGN COEFFICIENTS FOR STRUCTURE IN FIGURE 12a BASED ON R = 8

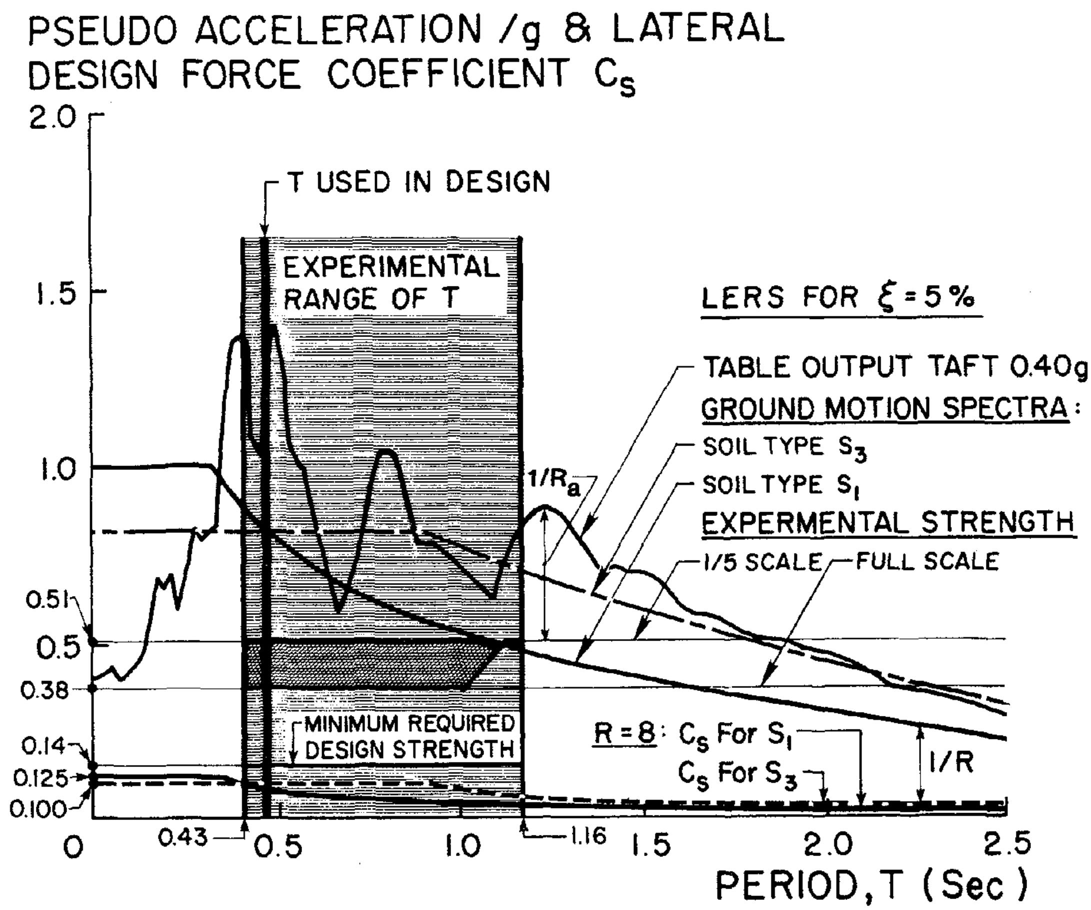


FIGURE 15 COMPARISON OF ATC MINIMUM REQUIRED DESIGN STRENGTHS, DESIGN STRENGTHS USED FOR EXPERIMENTAL MODELS, ATC 5% DAMPED LEDRS, 5% DAMPED LERS FOR SHAKING TABLE MOTION, AND MEASURED STRENGTHS

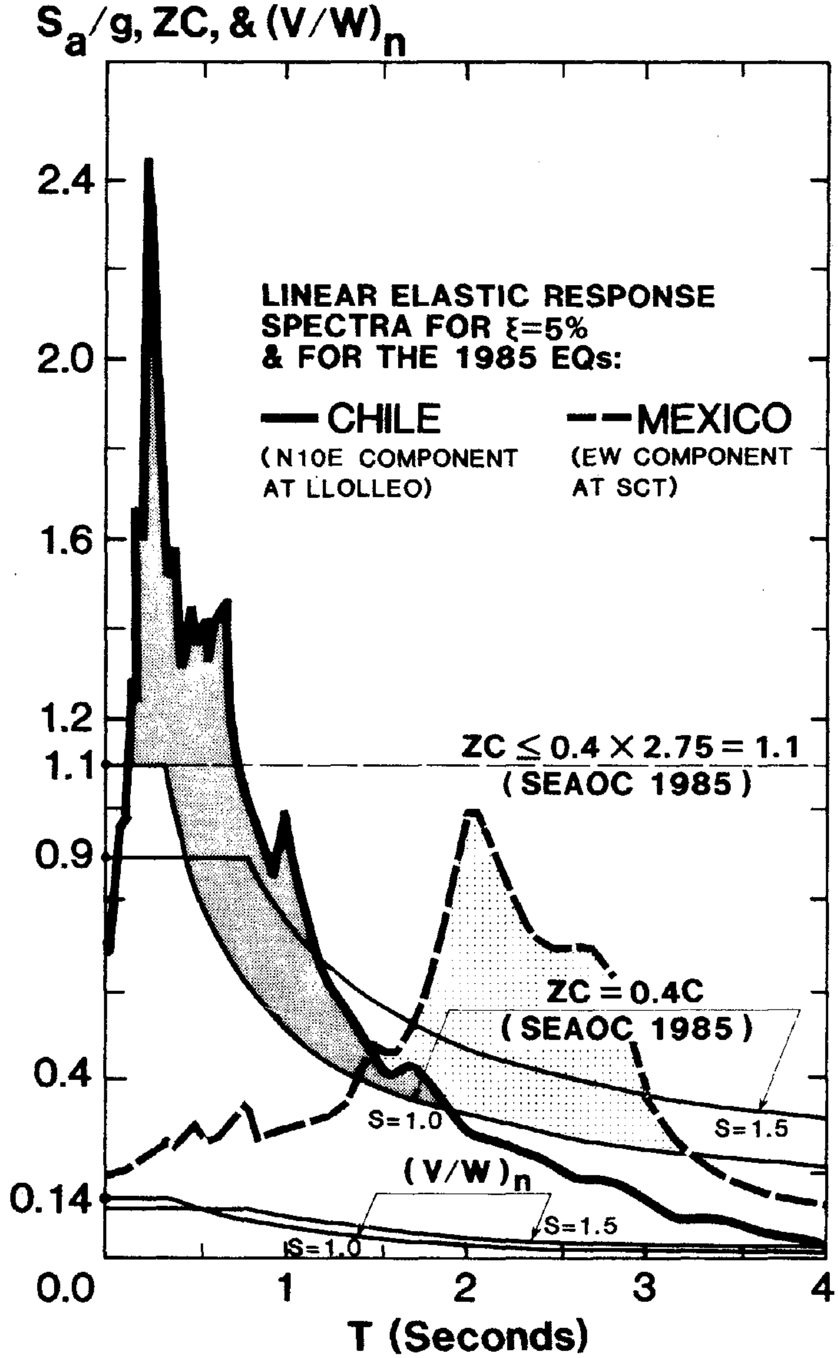


FIGURE 16a COMPARISON OF 5% DAMPED LERS FOR EW COMPONENT OF 1985 MEXICAN EARTHQUAKE RECORDED AT SCT (ZONE III) AND FOR N10E COMPONENT OF 1985 CHILEAN EARTHQUAKE RECORDED AT LLOLLEO ON FIRM SOIL WITH 1985 SEAOC 5% DAMPED LEDRS FOR FIRM SOIL (S = 1.0) AND SOFT SOIL (S = 1.5) AND WITH CORRESPONDING PRESCRIBED NOMINAL STRENGTH FOR REINFORCED CONCRETE DUCTILE MOMENT-RESISTING SPACES FRAMES ($(V/W)_n$)

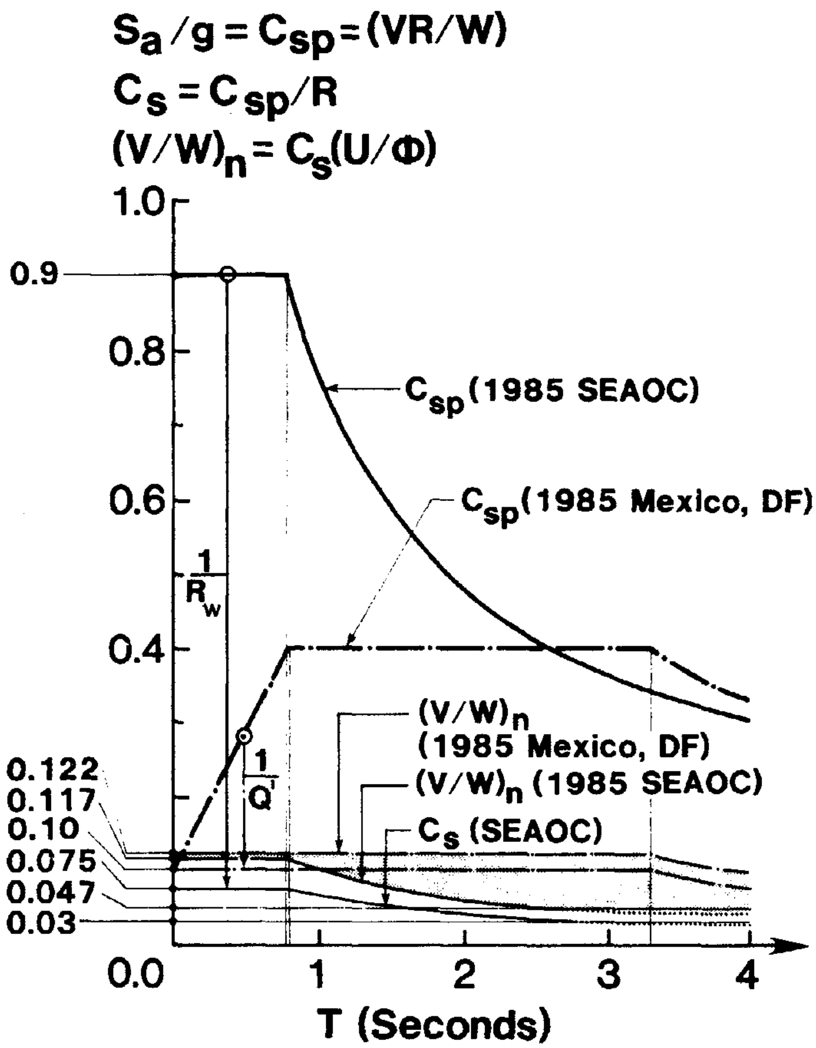


FIGURE 16b LINEAR ELASTIC DESIGN RESPONSE SPECTRUM (C_{sp}), DESIGN SEISMIC FORCES (C_s), AND MINIMUM NOMINAL FLEXURAL STRENGTH ($(V/W)_n$) SUGGESTED BY SEAOC IN 1985 FOR BUILDINGS OF OCCUPANCY CATEGORY III, HAVING A R/C DMRSF AND LOCATED ON SOFT SOIL (S_3 = 1.5) WITH VALUES ADOPTED IN 1985 MEXICO FEDERAL DISTRICT EMERGENCY CODE FOR GROUP B BUILDINGS IN ZONE III

EARTHQUAKE ENGINEERING RESEARCH CENTER REPORTS

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

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