CALIFORNIA INSTITUTE OF TECHNOLOGY
EARTHQUAKE ENGINEERING RESEARCH LABORATORY

EARTHQUAKE DESIGN CRITERIA FOR STRUCTURES

By

G. W. Housner and P. C. Jennings

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PREFACE

The earthquake resistant design of structures plays an important role in seismic regions both from the point of view of public safety and from the point of view of economical construction. Since construction is at a rate of approximately $15 billion per year in the seismic regions of the United States, unnecessary expenditures on earthquake protection could be very costly but, on the other hand, very costly damage could result if a city has inadequate earthquake protection and is shaken by a strong earthquake. The loss of life in an inadequately protected city could be very large, as evidenced by the reported 700,000 casualties inflicted by the Tang-Shan, China earthquake of 28 July 1976. The engineering profession has the technical responsibility for the safe and economical protection against earthquakes. The prime consideration in achieving this protection is the formulation of proper earthquake design criteria. Because the time and place of occurrence of future earthquakes cannot be foretold, the earthquake forces to which a structure will be subjected during its lifetime can not be specified at the time it is being designed and, therefore, consideration must be given to the desired performance of the structure if it should be subjected to weak earthquakes which have a relatively high probability of occurrence, or to very strong earthquakes which have relatively low probability of occurrence. The earthquake design criteria must then be formulated so that the building is, indeed, capable of the desired performance, and this formulation should be the responsibility of the project engineer.

Figure A. Damage to Olive View Hospital building during the 9 February 1971 San Fernando earthquake. The earthquake design criteria for this building were not proper for such strong ground shaking.

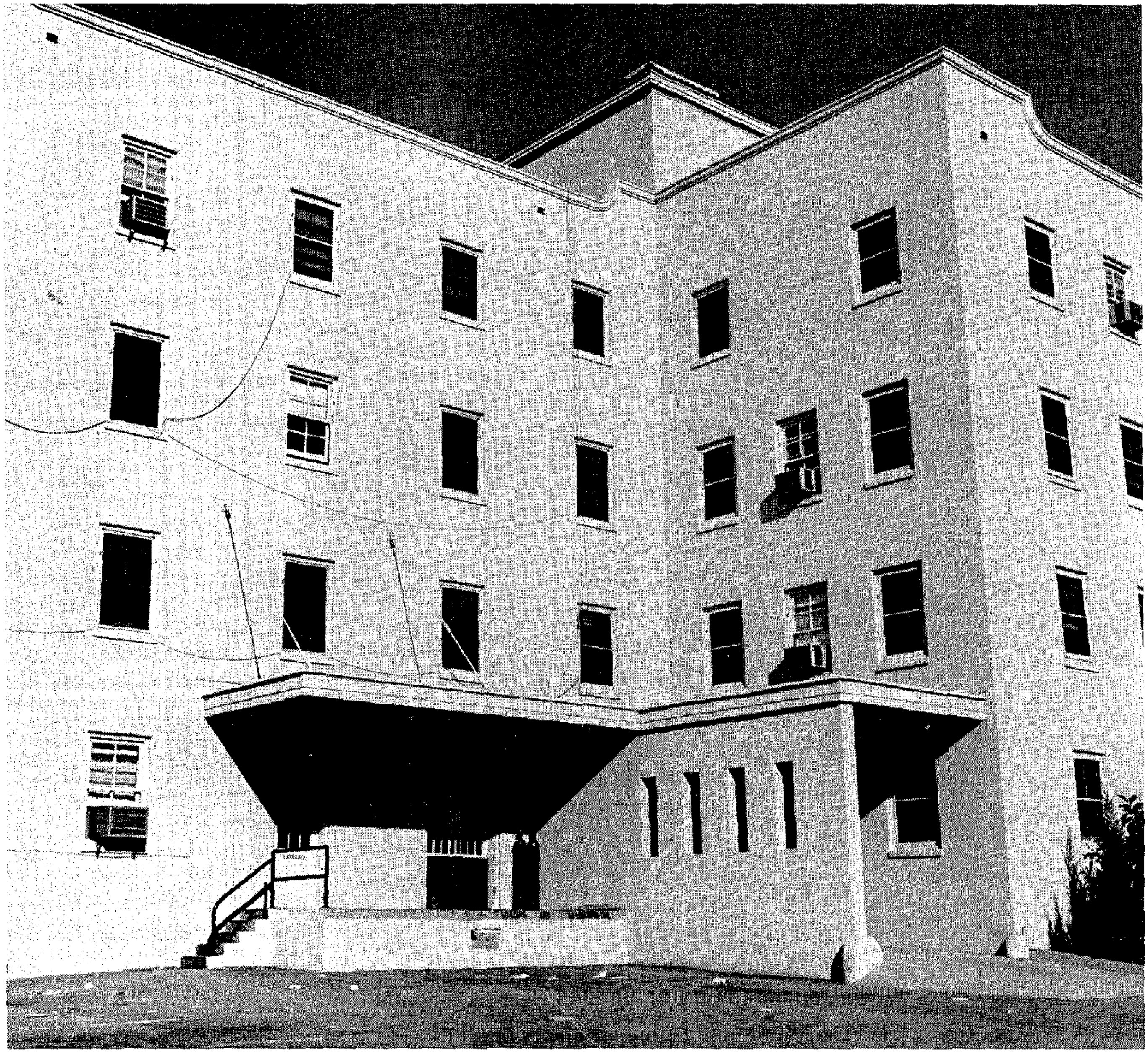

Figure B. Concrete building that survived without damage the strong ground shaking during the 9 February 1971 San Fernando earthquake, Veterans Administration Hospital. The seismic design criteria for this building were proper for the very strong ground shaking experienced.

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1. INTRODUCTION

When special projects are to be designed to resist earthquakes certain problems are encountered which do not arise when ordinary buildings are to be designed. In the latter case, the design is carried out in accordance with the requirements of the applicable building code and this relieves the engineer of the necessity for making judgemental decisions about the strength the structure should have, the ductility it should have, etc. The building code, in effect, represents a consensus of how ordinary buildings should be designed. On the other hand, special structures, because of the cost, the potential hazard, the need to maintain operations, etc., require special consideration. For example, attention should be given to how the structures or facilities will perform during future earthquakes; what is acceptable infrequent-damage; how much should be invested in providing earthquake resistance; will the design be approved by an outside review? Such questions are certain to arise when designing high-rise buildings, large dams, nuclear power plants, long span bridges, oil refineries, LNG storage facilities, offshore drilling platforms, chemical process facilities, port and harbor facilities, and other similarly complex and costly installations. It is very important that correct earthquake engineering decisions be made for such projects, from the standpoint of safety as well as of cost. These decisions should be made by the project engineer, for he is responsible for the engineering design and he is the only person having the necessary overall view of the project. It is not proper for the project engineer to permit the decisions to be made by the owner, the consultants, or the architect for they do not possess the knowledge and experience that is required.

The Problems of Design

In the earthquake resistant design of major projects two types of problems are encountered. The first are problems of a purely technical nature, which include the determination of the desired strength of the structure, the choice of structural type and material, the method of framing, the allowable stresses and strains, and the many details that comprise the engineering design process from its inception to the final structure. The second kind are more managerial in nature; these include the coordination of the contributions of consultants, such as geologists, seismologists and earthquake engineers, and the presentation and defense of the project and its earthquake resistant design before various governmental bodies and regulatory agencies, including the preparation of backup documentation. The second type of problem was at one time unimportant, but in recent years the activity in this area has increased greatly and some large projects now must receive approval from as many as 50 different political or regulatory bodies. A sizeable fraction of the attention of senior project engineers is devoted to this aspect of the project; and in some instances it seems to assume even greater importance for the success of the project than does the engineering design itself.

The prime technical problem in the earthquake-resistant design of a major project is the formulation of the design criteria, although the subsequent engineering analysis and design may also be difficult. When formulating the criteria it is necessary to keep in mind that, fundamentally, they are a means of specifying the desired aseismic capacities of the structures and facilities. The objectives of the criteria are twofold: First, to provide levels of earthquake resistance for the various parts of the project that are consistent relative to each other; and, second, to provide an absolute level of earthquake resistance that is appropriate to the desired performance of the project.

On the non-technical side, the requirements of coordinating the technical specialists and the involvement with regulatory agencies and political bodies can place a heavy burden on the project manager. In order to do his job effectively he must ensure good communication between geologists, seismologists, earthquake engineers, and designers, that is, between those who contribute information upon which the design criteria are based and those who will utilize the information. In the past, difficulties have arisen because of misunderstandings, particularly between engineers and seismologists whose training and experience predispose them to look at the earthquake problem differently. It is also essential for the project manager to have a good overall grasp of the various aspects of the earthquake design problem, for he must assess the conservatism, or lack of conservatism, in the final design and must arrange efficient interaction with regulatory and political groups.

Function of the Design Criteria

The primary function of the design criteria is to restate a complex problem, that has unknowns and uncertainties, into an unambiguous, simplified form having no uncertainties. The design criteria should provide clearly stated guidelines for the designers.

For example, when actually designing a structure, an engineer needs to know the forces and deformations that the structure should be able to resist. Some of these forces, such as gravity dead-loads are completely known but others that result from the transient action of nature or man, such as earthquake, wind or live-loads, are not known. This lack of knowledge must somehow be circumvented and a precise, unambiguous statement of the design conditions must be given to the design engineer. This is accomplished by means of the design criteria. The designer also needs to know the properties of the materials and structural elements that will be used, but as these are not precisely known, mainly because of imperfections in materials and workmanship, the design criteria must also take this into account. In the preparation of the design criteria allowance must be made for the uncertainties, and it is necessary to be cognizant of all the unknowns for which allowance must be made.

The traditional engineering design criteria for gravity and live-loads, for example, those in the Uniform Building Code, specify design loads that are greater than the actual loads typically encountered, and specify allowable design stresses that are appreciably less than the expected ultimate strength of the material. The purpose of this procedure is to ensure extra strength that is sufficient for unforeseen variations in loads, in material properties, and in workmanship. These criteria, in effect, tell the design engineer: "if you design according to these requirements the structure will be considered adequate." A similar approach can be taken for earthquake resistant design if the conditions are more or less the same for all projects.

However, if the seismic hazard varies markedly from place to place, and if structures vary in importance, cost, length of life, ease of repair, and consequences of failure, the formulation of seismic design criteria cannot, in general, be reduced to a simple rule of thumb, for then special knowledge and judgment are required.

2. USE OF SEISMOLOGICAL AND GEOLOGICAL DATA

When designing for a seismic region it would be very helpful to know exactly the ground shaking that the structure under consideration will experience during its lifetime. This foreknowledge, however, is not available, so recourse must be had to estimating what might happen in the future. Seismological and geological data form the bases for estimating future ground shaking at a site. The seismic history of a region, by showing what has happened in the recent past, gives a clue as to what might be expected in the near future. In this sense the past predicts the future, but the reliability of the prediction depends upon the quality and quantity of available data. Earthquake data of high quality, by definition, have instrumentally determined magnitudes and epicenters of all significant events. Earthquake data of satisfactory quantity would, by definition, include a sufficiently large number of events so that enough earthquakes of larger magnitude are included.

Earthquake Magnitude

In practice, the size of an earthquake is denoted by its assigned magnitude. In its original sense, the magnitude of an earthquake is the logarithm to the base ten of the maximum amplitude of the response of a standard seismograph at 100 km from the center of the earthquake; the seismograph having a natural period of 0.8 seconds and 80% of critical damping.* In a more useful, but less precise, sense the magnitude is a number that describes the size of an earthquake, that is, it describes the seismic energy released by the fault rupture and it describes the size of the area affected by strong ground shaking. (The imprecision of the magnitude scale when used in this sense is described in Appendix A). In practice, the magnitude of a large earthquake is based on a measurement made at a distance of hundreds of miles and which, therefore, does not contain any direct information about the nature of strong ground shaking near the causative fault. However, it is customarily assumed that two earthquakes having the same magnitude number will have similar ground shaking, other things being equal; but it should be kept in mind that other things are seldom exactly equal.

The adequacy of seismological data depends upon having a sufficient number of data points in the historical record, with magnitudes and locations determined, so that large magnitude events are also included. For example, if the data include only earthquakes having $M \leq 5$ the probability distribution would not be defined and it would be of questionable reliability to extrapolate to the probability

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*This definition now applies only to the local magnitude; M_L. There are currently several magnitude scales in use, and in the case of large earthquakes the commonly reported magnitude is not M_L, but is the surface wave magnitude, M_S, or the magnitude determined some other way.

of earthquakes of $M \geq 8$. Lacking sufficient data to define a probability distribution, it is customary in U.S. practice to assume a frequency distribution for M that is consistent with the seismic history of California, even though this introduces a degree of uncertainty.

In the less seismic regions of the U.S., the seismological data are relatively few and of poor quality. For example, in the eastern part of the country the available information on damaging earthquakes seldom includes the instrumentally determined magnitude of the event but instead gives Modified Mercalli Intensity (MMI) numerals. The MMI is of lower statistical quality than the magnitude, not only because it is based on personal observations of earthquake effects instead of instrumental records, but also because the actual interpretation is often unreliable. For example, a review of the effects of the 12 August 1929 Attica, New York, earthquake indicates a maximum MMI of VII instead of the VIII originally assigned to it (Fox and Spiker, 1977). The uncritical use of MMI data introduces a degree of uncertainty which may lead to an overestimation of seismic hazard.

Required Seismological and Geological Information for Design

The seismic history of the United States is not very long, being only two hundred years, more or less, depending on location and this is a short time for earthquake occurrence. This relatively short-time information can be supplemented by geological information about long-time tectonic processes that are measured in many hundreds or thousands of years. For example, faults that can be identified as having experienced slip during the past hundreds, or past thousands, of years can be taken to contribute to the seismic hazard of the region, but it is difficult to quantify this contribution. Various procedures have been employed to interpret the seismic hazard posed by such identified faults. The crudest approach is that which assigns a Maximum Capable Earthquake (MCE) to the fault (the MCE is sometimes called the Maximum Credible Earthquake but this latter name is so ambiguous and poorly defined that it is best avoided). For example, a fault whose discernible length is approximately 40 miles might be assigned a MCE of Magnitude 7, or one with a discernible length of 15 miles might be assigned a MCE of Magnitude 6.5. The MCE by itself is not a very informative number, for it does not distinguish between a fault that will have events of the approximate size of the MCE once per 200 years and one for which the return period is once in 500,000 years, even though this information would be very important to engineers preparing seismic design criteria.

A project manager should require the geological and seismological consultants to address the question of probability of occurrence. He should not accept a report that merely states "the recommended design earthquake is Magnitude 7.5," not only because it gives no indication of frequency of occurrence but also because it implies that the geologist-seismologist has made a decision about engineering design, which is outside his area of competence. The field of expertise of geological and seismological consultants is related to geologic and seismic hazards and their reports should describe the possible earthquakes together with estimates of probability of occurrence, or the possible intensity of ground shaking together with estimated probability of occurrence. The incorporation of the information into the design criteria should be the responsibility of persons who understand engineering design and performance of structures.

Strong motion accelerograms recorded in the past illustrate the kind of ground motions to be expected in the future, and the ground motion to be considered in the design can be exemplified by three components of ground acceleration which are consistent with recorded accelerograms. The recommendations of a seismological consultant should, preferably, present ground accelerations in the form of appropriate recorded accelerograms from particular earthquakes, or synthesized accelerograms that have appropriate intensity, duration, and frequency characteristics. The seismological consultant should also give the estimated probability of experiencing ground shaking that exceeds this accelerogram in severity. For example, it would be appropriate for him to give either the ground motion that would be exceeded once in 50 years, or the motion that would be exceeded once in 200 years, so long as it is properly identified; but it would not be acceptable to give a ground motion without identifying the expected frequency of exceedance.

Sometimes seismological consultants do not present accelerograms but instead give a less complete description of the ground motion. Properly, this less complete description should include a) the intensity of ground shaking, b) the duration of strong ground shaking, c) the frequency characteristics of the expected motion, and

d) the frequency of occurrence. The intensity of the ground shaking indicates to the engineer how severely his structures will vibrate; the duration of strong ground shaking indicates the degree of damage to be expected if the structure is stressed beyond the elastic limit; the frequency characteristics of ground motion should be identified, for earthquakes in different parts of the world may have different frequency characteristics and therefore can have different effects on structures; the estimated frequency of occurrence of ground shaking indicates the conservatism of the recommended ground motion.

Often the intensity of ground shaking is described by giving a value of peak acceleration, but by itself this is an ambiguous description, for two ground motions having the same peak acceleration can have appreciably different intensities so far as structural response is concerned. A much better method of describing the ground motion would be to compare it to a known accelerogram, such as Taft 1952 or a synthesized accelerogram. The description could thus be phrased as: 1.5 times as intense as Taft 1952, duration of strong shaking 1.2 times as long, and frequencies of motion all greater by a factor of 1.3. When the information is presented in this manner the engineer will understand what the seismologist means. More information can, of course, be given, but if any less information is given, the meaning will be ambiguous. Sometimes the seismological consultant describes the ground motion by recommending a smooth "design spectrum." This, however, is not proper, for a "design spectrum" is not the same as a "response spectrum" of actual ground motion or a smoothed "average spectrum," and it is precisely this difference that involves engineering judgment. For example, if the top 25% of the highest peak on an accelerogram is lopped off it would, in general, have very little effect on the response of structures and, therefore, when an engineer selects a smooth design spectrum based on an accelerogram, the zero-period spectral acceleration (sometimes called "effective acceleration") may, with justification, be smaller than the peak ground acceleration. If the structure to be designed is highly ductile, the project manager may set the entire design spectrum at a lower level than the response spectrum. The validity of thus specifying the design spectrum depends on knowing how to correlate the spectrum with the properties of the structure to be designed.

3. ENGINEERING CRITERIA

The Design Spectrum

The central feature of most earthquake-resistant design criteria is the design spectrum; an example of which is shown in Figure 1b. The jagged response spectrum (Figure 1a) describes the computed response of different oscillators to a particular ground motion whereas the smooth design spectrum is a specification of the level of seismic design force, or displacement, as a function of natural period of vibration and damping level. Implicit in Figure 1b is the condition that the level of force prescribed by the design spectrum is to be associated with a specified level of material resistance, for example, the allowable design stresses or strains. The resultant

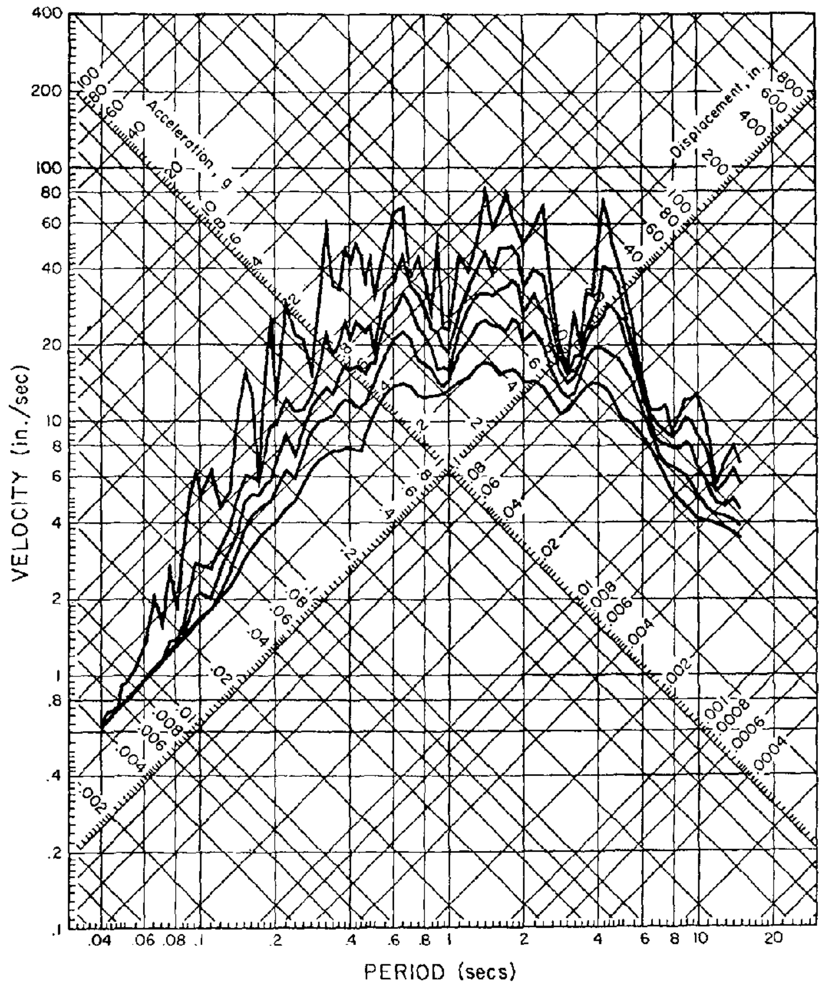


Figure 1a. Response spectrum of NS ground motion recorded at Holiday Inn during 9 February 1971 San Fernando earthquake (0, .02, .05, .1, .2 fraction of critical damping).

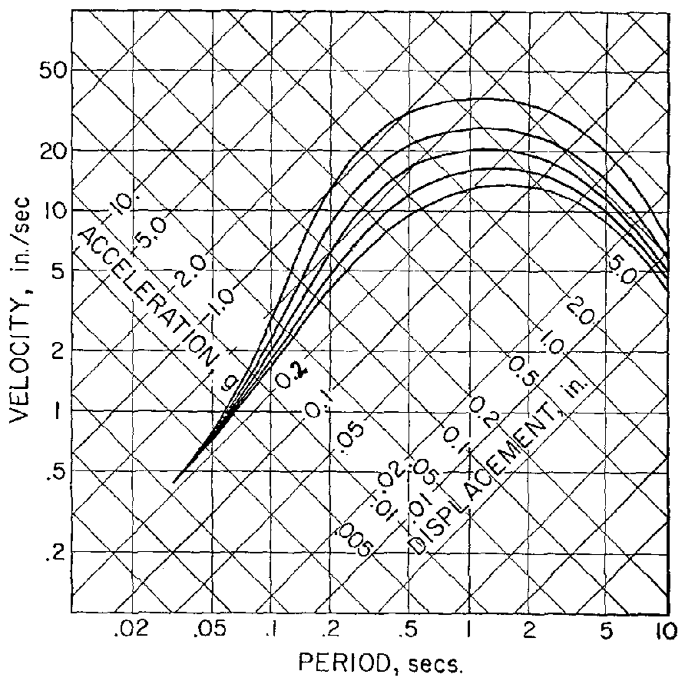


Figure 1b. Example of smooth design spectrum based on Figure 1a. (0, .02, .05, .1, .2 fraction of critical damping).

effect is, thus, a specification of the required earthquake resistance of the structure and its elements. If the material resistance is stated in terms of allowable stresses, the design spectrum is a specification of the strength of the structure; if the material resistance is expressed in terms of permissible ductile strains the design spectrum becomes a specification of the capacity of the structure to deform, that is, its ductility.

Four factors combine to specify the capacity of structures to resist earthquakes. These are: 1) the level of the design spectrum; 2) the designated spectral damping; 3) the allowable design stresses and strains; and 4) the method of determining the natural periods of vibration of the structure. It is customarily considered that the level of the design spectrum is the most important, but the other three factors together can be equally or more important. For example, Figure 2, which is a replotting of the design spectrum in Figure 1b, shows the effect that damping has on the design acceleration. The effect is very strong for damping less than 0.1 of critical and periods less than 2 seconds. Table I shows that the same spectral acceleration (.54g) is obtained when the spectrum level is .21g, or when it is 25% larger or 25% smaller, if suitable adjustments are made of the other parameters.

Table I

Spectrum level at zero period (g)	.16	.21	.26
Damping	.03	.04	.05
Natural period (sec)	.35	.4	.5
Allowable stress or strain	.9	1	1.2
Spectral acceleration (g)	.54	.54	.54

In setting the design spectrum, the project manager should take into account the acceptable degree of damage and the likelihood of its occurrence. He must also consider the actual capacity of the structure that results from the use of the design spectra and the specified capacities of the materials of construction. This is obviously a problem requiring both engineering knowledge and judgment and, because of the complexities and uncertainties, considerable reliance

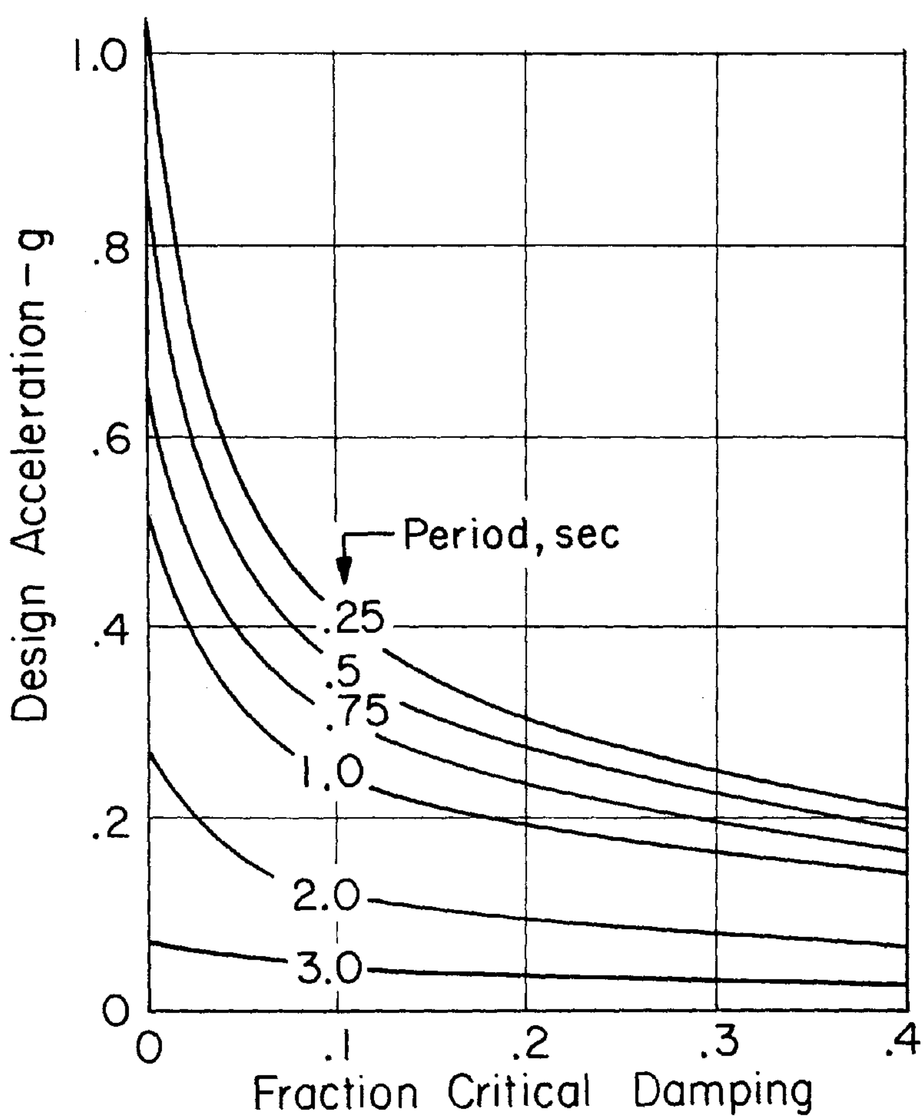


Figure 2. A replot of the design spectrum shown in Figure 1b. This diagram exhibits the effect of damping on spectrum values.

must be placed on knowledge of how engineered structures have performed in past earthquakes. For example, the San Fernando earthquake provided evidence that the level of design, together with the material resistances and the quality control specified in California's Field Act ensure the successful performance of typical one- and two-story school buildings during very strong ground shaking (Hudson and Jephcott, 1974). The buildings performed successfully even though the nominal levels of design accelerations were much lower than the actual ground accelerations.

The Capacity of Structures

The apparent paradox that the code value of acceleration for which a structure was designed is much smaller than the recorded peak acceleration of the ground motion that the structure successfully survived, can be explained without recourse to such terms as "effective peak acceleration" and "sustained peak acceleration" which are smaller than the peak acceleration itself. The explanation is that the allowable design stresses and strains in the building code are not indicative of the material and structural resistances under dynamic conditions. To clarify this, it is necessary to establish the true relation between the actual dynamic capacity of engineered structures and the levels of the basic components of the design criteria: spectrum level, damping and allowable material resistance.

An example of this relation is shown in Figure 3, which is derived from the San Fernando earthquake, an accelerogram of which is shown in Figure 4. Figure 3 shows three sets of data for multi-story reinforced concrete structures constructed since

1964, plotted as a function of the fundamental periods of the buildings measured during the earthquake. Two of the buildings are shown in Figures 5 and 6 . The triangular data points, the top group, are the maximum acceleration measured on the roof of the buildings. The circular data points represent the maximum base shears experienced during the earthquake by the fundamental modes of vibration of the structures. The base shears were determined from the computed maximum displacements reported by Hudson, et al. (1969-76), and from a knowledge of the distribution of mass in the buildings and the shapes of the fundamental modes. The ticked circles indicate that this level of base shear was associated with some structural damage. It should be noted, however, that none of the structures represented in the figure were dangerously damaged and all could have resisted significantly stronger shaking without collapse. The square data points in the figure are the base shear values employed in the designs; these were determined by the designer in accordance with the applicable building code. The significance of Figure 3 is that it indicates that such structures on the average, can be expected to resist base shears that are two-to-three times larger than the code design values without severe structural damage. The margin of safety against collapse of these structures was not tested by the San Fernando earthquake, but the data suggest that, on the average, responses equivalent to five or more times the design base shear could have been resisted without collapse, though severe damage would probably have resulted.

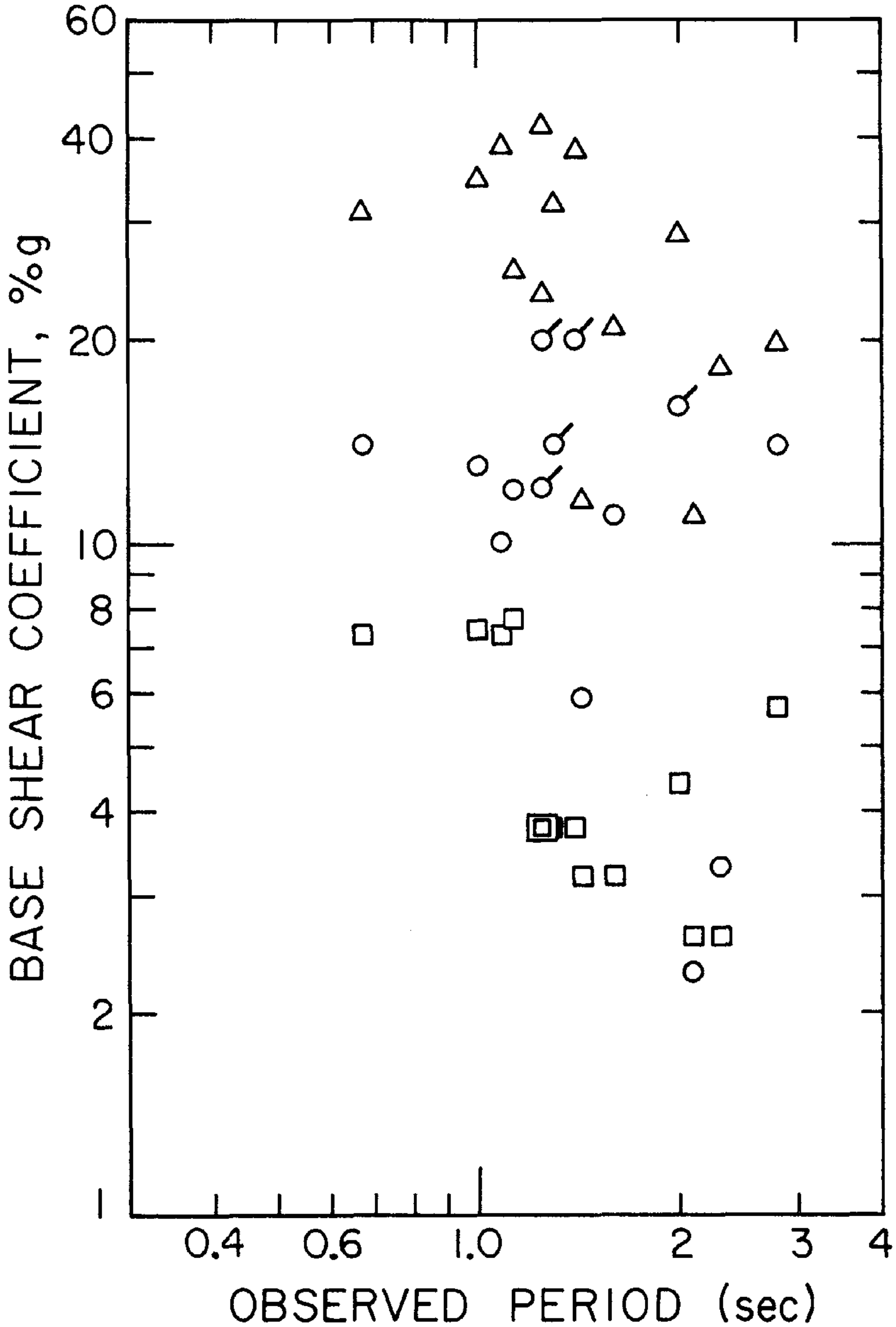


Figure 3.

Base-shear coefficients of multi-story concrete buildings that experienced strong ground shaking during the San Fernando earthquake. The triangles represent the maximum acceleration recorded on the roof; the circles represent the maximum first-mode base shears experienced during the earthquake (the ticked circles represent structures that had some damage); squares represent the nominal code values of base shears for which the buildings were designed. The periods are those of the fundamental modes of vibration. (Base shear coefficient is equal to base shear divided by total weight of structure).

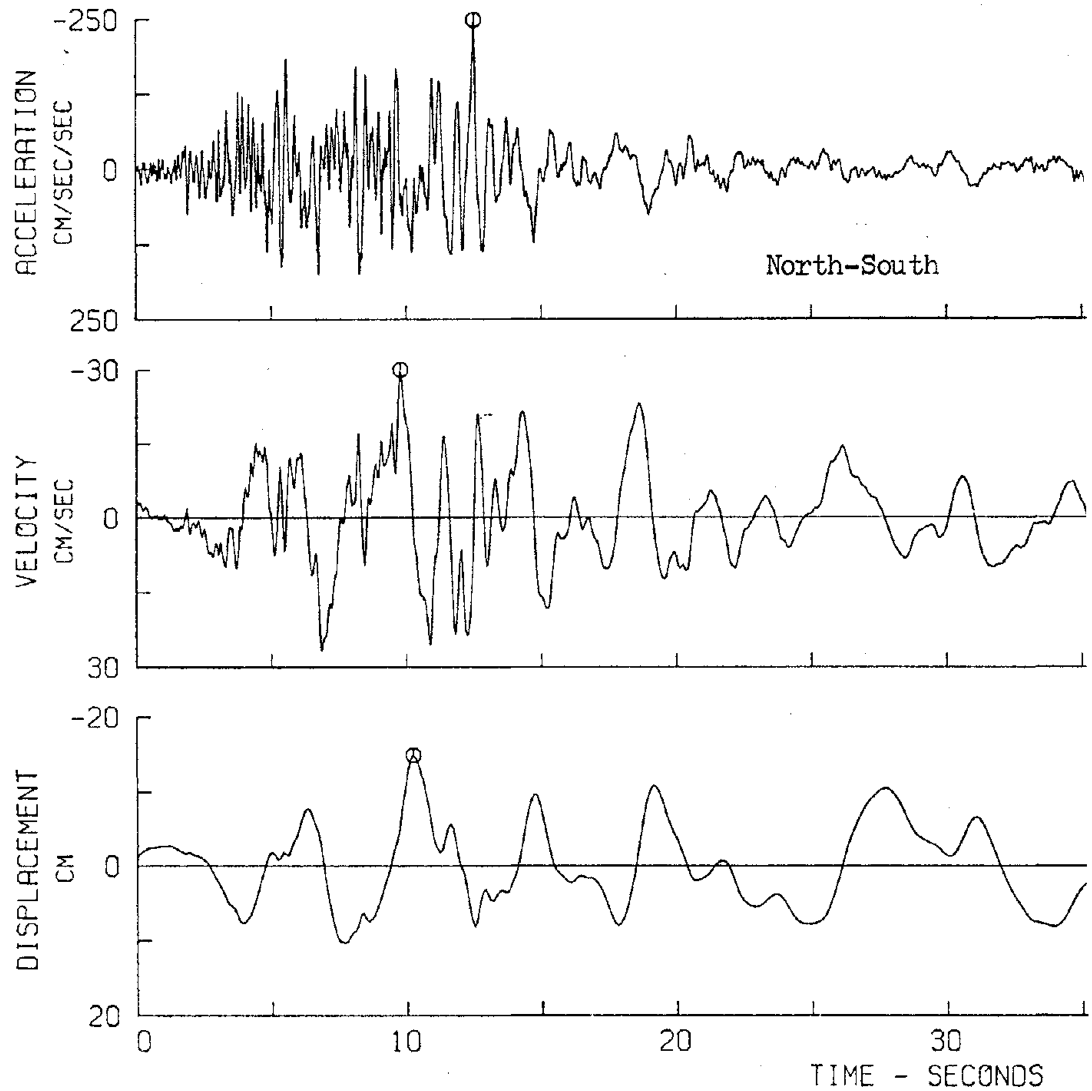


Figure 4. Recorded ground acceleration and computed velocity and displacement at Holiday Inn, approximately five miles south of the nearest portion of the causative fault.

Figure 5. Holiday Inn after the San Fernando earthquake. This seven-story concrete frame building underwent severe cracking of beams and columns which was later repaired with epoxy. The peak ground acceleration was 26%g and the spectral acceleration from Figure 1a for 0.8 secs. and 5% damping is 0.4g. The first-mode base shear for a multi-story frame building is given by spectral acceleration multiplied by about .75 times the weight of the building. (Base-shear $\approx$ 0.75 x weight x spectral acceleration).

The capacity of buildings to resist strong ground shaking is illustrated from another viewpoint in Table II. The table describes, for California conditions, the expected performance of buildings of different types to the potentially damaging shaking that can occur in major earthquakes. The table is not meant as a quantitative guide to the assessment of hazard, but rather as a first approximation to the expected effects of strong ground motion.

If the observed ability of structures to resist earthquakes is not taken into account when formulating the design criteria it is possible to end up with inconsistent results. For example, as shown in Figure 7 several concrete buildings with 8 inch thick shear walls survived, without damage, the severe ground shaking at the Veterans Administration Hospital during the San Fernando earthquake. However, the seismic design criteria for the new, post-earthquake, Olive View Hospital building, at a site adjacent to the VA Hospital, were so stringent that these VA buildings could not satisfy them.

Modifications in the Shape of the Spectrum

In order to specify consistent levels of structural capacity for buildings having different natural periods, the shape of the design spectrum should reflect the relative intensities of expected motions at different frequencies. Because the energy in the site ground motion at shorter periods would be dominated by nearby earthquakes of moderate size rather than by more distant larger shocks, and because the longer period energy would be dominated by large earthquakes even if occurring on more distant faults, the shape of the design

Table II

Expected Degree of Damage vs. Intensity Based on
San Fernando Earthquake Shaking
for Southern California

Class of Building	Intensity of Ground Shaking			
	15-20% g	20-30% g	30-40% g	40-50% g
A - Above average modern building	None	Minor or none	Moderate	Severe
B - Average modern building	Minor or none	Moderate	Severe	Major
C - Below average modern building	Moderate	Severe	Major	Partial collapse
D - Old, pre-code building	Moderate to severe	Major	Partial collapse	Partial collapse

Minor Damage: can be repaired without appreciable interference with normal operations.

Moderate Damage: can be repaired with small interference with normal operations; perhaps equivalent to closing down for several days.

Severe Damage: significant damage to structural members; repairs will require closing for at least several weeks.

Major Damage: extensive damage to structural elements; repairs require closing down for several months.

Partial Collapse: repairs require closing down for an extended period, from five months to a year. For Class D structures the building may have to be abandoned.

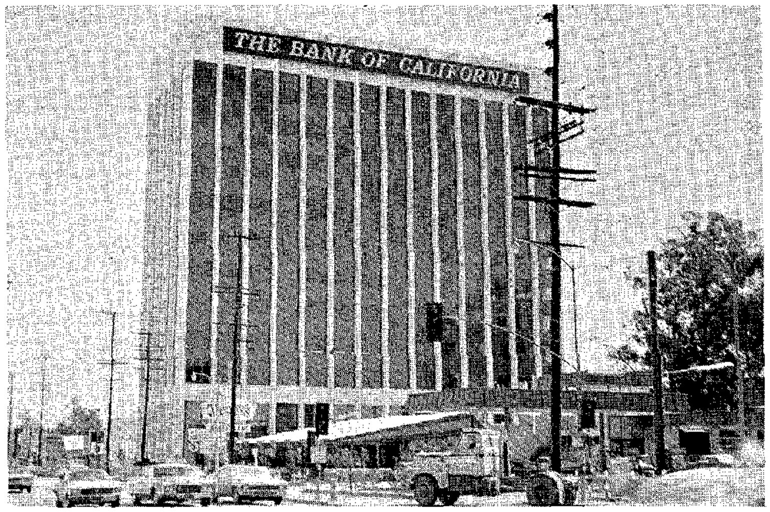
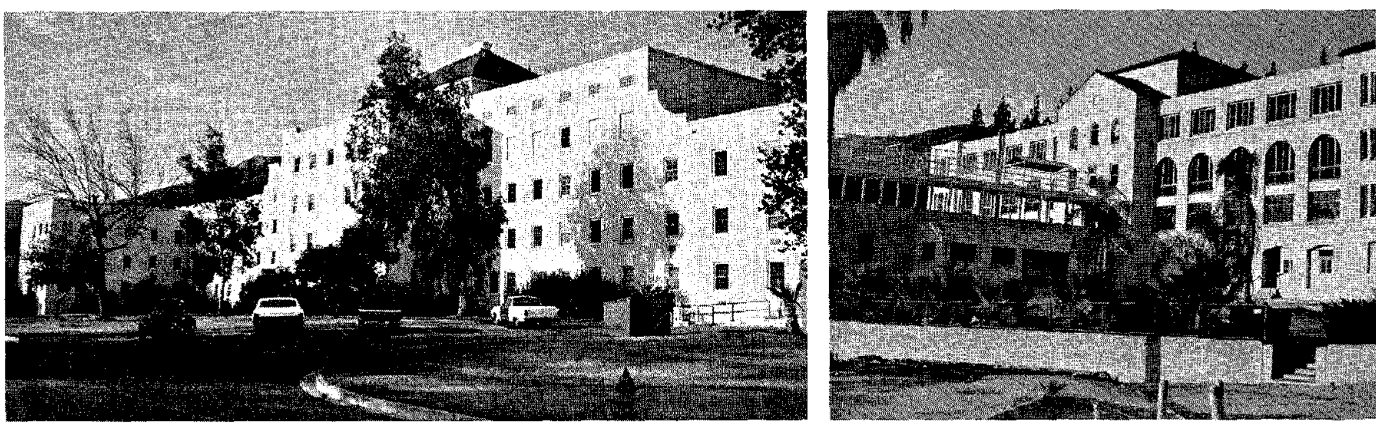


Figure 6. The 12-story California Bank Building after the San Fernando earthquake. This concrete frame building experienced appreciable cracking of beams and columns. The peak ground acceleration during the earthquake was .23g and the maximum base shear in the first-mode was 3.2 times larger than the nominal code value of base shear. (N 11° E motion)



(a) (b)

Figure 7a, b. Two concrete shear wall buildings at the Veterans Administration Hospital that survived the San Fernando earthquake without structural damage. It is estimated that the ground motion was approximately twice as intense as at Holiday Inn (approximately 50%g peak value). This would correspond to a spectral acceleration of 1.2g at a period of .4 secs. and 5% damping. The eight inch thick concrete walls resisted the earthquake forces without damage.

spectrum may be dependent to a degree on the expected occurrence of earthquakes in the region of the site. When formulating the design criteria, however, it is not justified to go to great lengths in tailoring the shape of the design spectrum to fit hypothetical earthquake hazards, for the present state of knowledge does not warrant this. It is recommended that such modifications be limited to simple and relatively minor alterations to a spectrum of standard shape.

A related problem arises concerning the adjustment of the design spectrum to accommodate possible influences of local geology and soil conditions. Unfortunately, there is a lack of data bearing directly on this problem, and such data accumulate slowly because of the cost of necessary instrumentation and because of the infrequent occurrence of strong earthquakes. To throw light on the possible effects of local soil conditions, special computations are often made which involve estimating the ground motion at depth (bedrock or firm soil), and then propagating this motion to the surface through linear, nonlinear, or iteratively linear models of the overlying soils. The seismic waves are assumed to be planar, horizontal shear waves that propagate vertically. Such analyses are often made for major projects sited on relatively soft soil. They can give useful insight if the actual geological and seismological conditions do not differ greatly from the conditions postulated by the computational procedures, as was the case, for example, of the well-known recorded behavior of the soft soil in Mexico City. However, it is very difficult to assess the differences between the assumed and the actual conditions and in practice this approach has sometimes been misused. The inconsistencies that result in trying to assess potential site effects could be reduced by making careful comparisons of predicted results with existing accelerograms that were recorded under similar geological and seismological conditions. Such relevant accelerograms, if available, should be used as the primary guide in the adjustment of the shape of the design spectrum, and the results of analytical and computational studies should be used as secondary guides.

The Role of Statistical and Probabilistic Analyses

In earthquake engineering there are many situations where essential factors cannot be precisely defined because of a lack of information. For example, the physical properties of the concrete and steel are not known precisely to the engineer when he makes the design; and the quality of construction workmanship is not known. These could be made known to the designer by means of additional quality control, testing and inspection but the cost would be prohibitive, so the uncertainties are accepted. In this case, it is considered to be cost-efficient to accept and to deal with uncertainties rather than to try to eliminate them. In the case of earthquake ground motions, it is uncertain where and when earthquakes will occur and how large they will be, and it is not known what ground motions they will produce. Again, in principle, these uncertainties could be reduced to small levels, for the problem is solvable if the states of stress and strain-rate in the earth's crust were precisely known, and if the failure strength of the rock were known, and if all the relevant physical properties of the earth's crust through which the seismic waves travel were also known; and if ample time and money were available for computing. The difficulty and cost involved make it unlikely that the foregoing problem will ever be solved and it is necessary, therefore, to accept a lack of knowledge and to deal with it as best possible. Statistical and probabilistic analyses are tools for dealing with scientific ignorance. Though they cannot generate any new factual information not already included in the basic data, such analyses provide subtle and inconspicuous ways of making assumptions about the basic data which can provide useful guidance for decision making. Because of their subtle nature, however, assumptions that are injudicious can provide misleading guidance.

When formulating design criteria it is necessary to specify the ground motion which the structures must be designed to resist. As the characteristics of future ground motion are not known, it is necessary to utilize other information that bears on the problem, for example, statistical information on the past occurrence of earthquakes of various magnitudes. If it were known where earthquakes will occur in the future and how large they will be, estimation could be made of the ground shaking at the site. This knowledge, however, is not available, for what is known is where earthquakes have occurred in the past. The seismic history of a region of area A conveys some information about the future, but only in the sense that the seismic record of the past N years is a data sample that is more or less like the seismic events that will occur during the coming N years. The degree of similarity depends largely on the product (NA), there being little similarity for smaller (NA) and greater similarity for larger (NA). It appears that for the seismicity of the 160,000 square miles of California, a value of $(NA) > 3 \cdot 10^7$ square-mile-years (a period of 200 years) represents a reasonably high degree of similarity, and $(NA) < 3 \cdot 10^6$ represents a very low degree of similarity. The foregoing figures indicate that for regions having lower seismicity than California, or smaller areas, the seismic history will not bear close similarity to the corresponding future period of years, and in order to draw conclusions it is necessary to assume the shape of the long-time, frequency distribution of earthquakes in the area. This assumption permits statements to be made about probability of occurrence. A useful way of making probabilistic statements is to put them in a comparative form; for example, comparing with the seismicity of the State of California, which is reasonably well known. The probabilistic quantities for a special region can then be compared to those for California, e.g., the probability of exceeding ground shaking of a specified intensity in metropolitan Los Angeles is estimated to be p_1 which compares with p_2 calculated using the average seismicity for the State of California. Since much more is known about the occurrence of earthquakes in California than in other parts of the U.S., California should be taken as a reference point and estimates of seismicity for other regions could be better understood if comparison is made with California.

4. THE MARGIN-OF-SAFETY IN EARTHQUAKE-RESISTANT DESIGN

Earthquake-resistant design, like other engineering design, has the objective of achieving a functional and economical design in the face of imprecise knowledge of the forces that will act upon the structure. Because of the imprecision in knowing the loads and the need for a safe design, most structures have substantial margins-of-safety for the loads they actually receive in their lifetimes. They are overdesigned in the sense that lack of knowledge about earthquake forces and other loads they will be subjected to leads to a resistive capacity which will not be fully used. This situation can be described in statistical terms in which uncertainties about the intensity of earthquake shaking and the degree of earthquake resistance are stated in terms of probabilities of occurrence and probabilities of amplitudes of response. Since it is impossible to prove that hypothetical events will not occur, it is not possible to prove that a structure will have a zero probability of failure. The probability of failure can be reduced by the provision of extra resistance, but it cannot be made equal to zero. In other words, engineers may be able to design earthquake-proof structures, but they cannot prove it.

The situation described above affects the way conservatism is brought into earthquake-resistant design and the way the design criteria are presented to regulatory agencies. In building codes, and in some projects, conservatism in the design is implicit in the sense that the design criteria are established by subjectively taking into account the nature and levels of forces, the types and quality of construction, the properties of materials, and the experience of structures during earthquakes. A determination of the margin-of-safety of each particular item is not attempted and the overall margin-of-safety is not known.

In the case of major projects the problem should be approached explicitly; each feature of the problem should be examined separately and a decision made concerning the appropriate level. This approach has the advantage that each important aspect of the project is subjected to careful study by knowledgeable professionals and the chance of overlooking some major point is minimized. However, a difficulty is inherent in this approach, for unless the project engineer keeps an overall check on the procedure there is a danger of compounding the factor-of-safety in the sequence of decisions that lead to the earthquake-resistant design criteria. For example, if the geologist is 1.5 times conservative on the capability of faults in the area of the site, the seismologist 1.5 times conservative on the size of the design earthquake, the earthquake engineer 1.5 times conservative on the strength of expected shaking, and if the design engineer is 1.5 times conservative on the allowable material responses, the final conservatism compounds to 500%. Figure 8 shows the sensitivity of the overall factor of safety to the number of sequential steps and the individual factors of safety. Without some overall assessment of the conservatism, design criteria can become excessively conservative. There would, of course, also be a possibility of ending up with deficient criteria if the consultants were all underconservative,

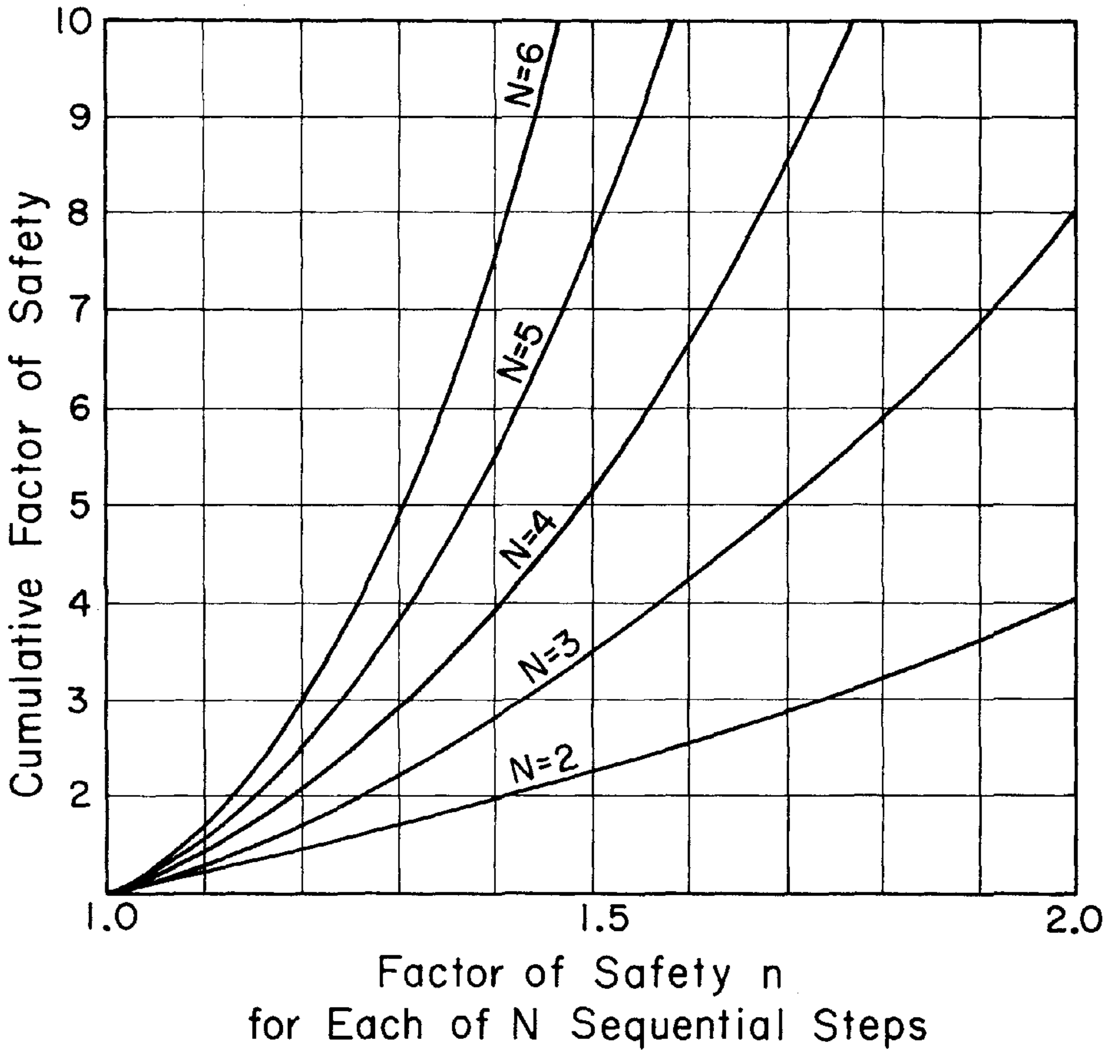


Figure 8. Cumulative factor of safety resulting from a sequence of N steps each having an individual factor of safety n. This diagram illustrates how the overall factor of safety can be escalated by imposing individual factors upon successive design steps. For example, if four successive factors of 1.5 are imposed, the cumulative factor of safety is 5.

but this seems to be a very remote possibility in the present climate.

Compounding of conservatisms can also occur when the design criteria are reviewed by regulatory agencies or political bodies. In the best of circumstances, the review panels are composed of knowledgeable people with access to consultants who are expert in various aspects of the earthquake problem. It is not usual, however, for any single panel member to have an overall view comparable to that of the project engineer, so the review tends to focus on those features of the problem that lie within the experience of the panel members and their consultants, and extra conservatism is introduced at these points without consideration of the conservatism in the other parts of the design criteria. Also, the most obvious way for a reviewing agency to show that a good job is being done is to require an increase in the design criteria. Furthermore, to a degree, the panel members and the consultants have their reputations at stake but they are not directly answerable for the cost of the earthquake protection; and, in many cases, the hearings and deliberations of the reviewing panel are open to the public, a feature which tends to emphasize the problems over the means and costs of providing solutions to the problems. Because of the foregoing aspects, the outcome of the regulatory process tends to be a very stringent set of earthquake-resistant design criteria; the end result tends to approach an upper bound of the judgments of all the individual parties involved, rather than a compromise value.

Another factor that should be considered when dealing with regulatory agencies, is the requirement to solve a problem in several ways. This arises because a regulatory body can demonstrate that it is doing its job by requesting information not included in the material under review. Thus, if one method has been used to determine earthquake-resistant design criteria, there is a tendency for the reviewing panel to ask how the result compares to that obtained by another approach, and then to require that the most conservative approach be used even though it has not been correlated with other conservatisms in the design criteria. Therefore, the project engineer should consider several approaches and be prepared to explain them, even if they are not used in setting the design criteria.

It is, of course, easier to comment critically on the review of seismic design criteria by regulatory bodies than it is to suggest alternative procedures. One step that would help, however, would be to involve more knowledgeable engineers in the reviewing process. The engineering viewpoint is always well-represented on the side of those applying to regulatory bodies, but it is often underrepresented on the reviewing panels themselves.

5. CONCLUSIONS

A project engineer, when faced with setting earthquake design criteria, should keep in mind that the criteria specify the desired performance of structures under future conditions. Because it is not possible to prove that hypothetical events will not occur in the future, it is not possible to formulate design criteria for zero probability of failure. Because of this the project manager should expect the recommendations of the geological, seismological and earthquake engineering consultants to be based on probability considerations which should be stated explicitly. The project manager cannot be an expert in geology, seismology and earthquake engineering so he must rely on the consultants; however, it is essential that he know the proper questions to ask. The following are examples of questions that should be asked:

1. What active faults are located within 50 miles of the site; particularly, what faults are close to the site, and in what sense are they active?

2. What significant earthquakes have occurred within 50 miles of the site? What were their characteristics?

3. What is the estimated frequency of occurrence of future earthquakes of various magnitudes in the general site vicinity?

4. What is the estimated intensity of ground shaking at the site that will be exceeded once per N years? (N may be one or more of the following: 50, 100, 200, 1,000, 10,000.)

5. What accelerograms, response spectra, or average spectra are representative of the above ground motions, in terms of intensity, duration, and frequency content?

6. What would be the consequences to the structures and facilities to be designed of various degrees of overstressing and straining beyond the elastic limits?

7. What would be an acceptable level of damage as balanced against probability of occurrence?

8. What ductility capability should the structure have, as balanced against the cost of providing it?

9. In view of the foregoing, what design spectrum should be used; what design-values of damping should be used; and what allowable stresses and strains should be used?

10. What resistive capability will the use of the design spectrum, the design damping, and the allowable stresses and strains actually provide?

Being aware of these questions and their answers, the project manager will be in a better position to make the necessary technical decisions and to guide the project through the regulatory processes.

6. REFERENCES

1. F. L. Fox and C. T. Spiker, "Intensity Rating of the Attica (N.Y.) Earthquake of August 12, 1929," Earthquake Notes, Eastern Section of Seismological Society, vol. 48, nos. 1 and 2, 1977.

2. D. K. Jephcott and D. E. Hudson, "The Performance of Public School Plants during the San Fernando Earthquake," Earthquake Engineering Research Laboratory, California Institute of Technology, 1974.

3. Proceedings of the Conference on Managua, Nicaragua Earthquake of December 23, 1972, Earthquake Engineering Research Institute, November, 1973.

4. Engineering Features of the San Fernando Earthquake, February 9, 1971, P. C. Jennings, ed., Earthquake Engineering Research Laboratory, California Institute of Technology, Pasadena, 1971.

5. San Fernando Earthquake of February 9, 1971, L. M. Murphy, Ed., U.S. Department of Commerce, Washington, 1973.

6. The Agadir-Morocco Earthquake of February 29, 1960, American Iron and Steel Institute, New York, 1962.

7. The Great Alaska Earthquake of 1964 - Engineering, G. W. Housner and P. C. Jennings, Eds., National Academy of Sciences, Washington, 1973.

8. Strong Motion Earthquake Data Reports, D. E. Hudson, Ed., Earthquake Engineering Research Laboratory, California Institute of Technology, Pasadena, 1975.

9. D. Foutch, G. Housner, and P. Jennings, "Dynamic Responses of Six Multi-Story Buildings during the San Fernando Earthquake," Earthquake Engineering Research Laboratory, California Institute of Technology, October 1975.

10. M. Sozen and R. Matthiesen, Engineering Report on the Managua Earthquake of 23 December 1972, Committee on Natural Disasters, Commission on Sociotechnical Systems, National Research Council, National Academy of Sciences, Washington, D.C., 1975.

7. APPENDIX I

Earthquake Magnitude

When a fault ruptures there is a sudden reduction of shear stress (stress-drop) at the fault plane which transforms static strain energy in the rock into stress waves. As the stress waves travel away from the fault they produce shaking of the ground surface whose intensity attenuates with increasing distance. Because of inhomogeneities in the earth's crust complex waves are produced, which include compressive waves, shear waves, Rayleigh waves, Love waves, etc. These waves approach a site on the surface of the ground from different directions, both in azimuth and in elevation, with the predominant transport of energy being away from the fault. In general, the larger the slipped fault area the greater is the amount of strain energy released and the larger is the surface area affected by strong shaking, and the larger is the "felt area" of ground shaking. Any measurement that characterizes the size of the area of strong shaking, or the size of the "felt area," could serve as an indication of the "size of the earthquake." C. F. Richter's earthquake magnitude scale uses, as the pertinent measurement, the peak amplitude recorded by a standard Wood-Anderson seismograph, which has a natural period of 0.8 seconds, approximately 80% of critical damping, a magnification of 2800, and is located 100 kms distant.* The peak amplitude, A, of

*This instrument can be compared with the standard seismoscope which has a natural period of 0.75 seconds and 10% of critical damping, and could also be used for magnitude determinations.

Wood-Anderson seismograms varies over the surface of the ground in a manner similar to the variation of intensity of ground shaking, being small at large distances from the fault and thousands of times larger close to the fault; so for a measure, the $\log_{10}(A/A_0)$ is used, A_0 being a constant. A schematic plot of $\log(A/A_0)$ for an earthquake is shown in the accompaning diagram (Figure 9) where it is seen that the contour lines of constant values are rather irregular oblong curves. The plot of $\log(A/A_0)$ forms a hill-shaped surface and it is clear that the volume of the hill, $M_V = \int_{-\infty}^{+\infty} \int_{-\infty}^{+\infty} \log(A/A_0)\,dx\,dy$ would be a good measure of the size of an earthquake, but it would be impractical to evaluate. A less precise, but more practical, measure was defined by Richter

$$M_L = \log(A^*/A_0)$$

$$A^* = \text{amplitude at } 100 \text{ km}$$

$$A_0 = 10^{-3} \text{ mm} = A^* \text{ for } M = 0$$

Actually, modern seismographs are not Wood-Andersons and are not at 100 km from the center of the earthquake, but seismologists can correct for instrument characteristics, and for distance, to obtain an equivalent M. Because of the noncircular shape of the contour lines of $\log(A/A_0)$, two different seismographic stations will not, in general, compute the same value of M_L, and the "official" value is usually the weighted average of several. A more stable measurement would be one based on the spectrum of the seismogram rather than on the peak amplitude but the work involved renders this impractical.

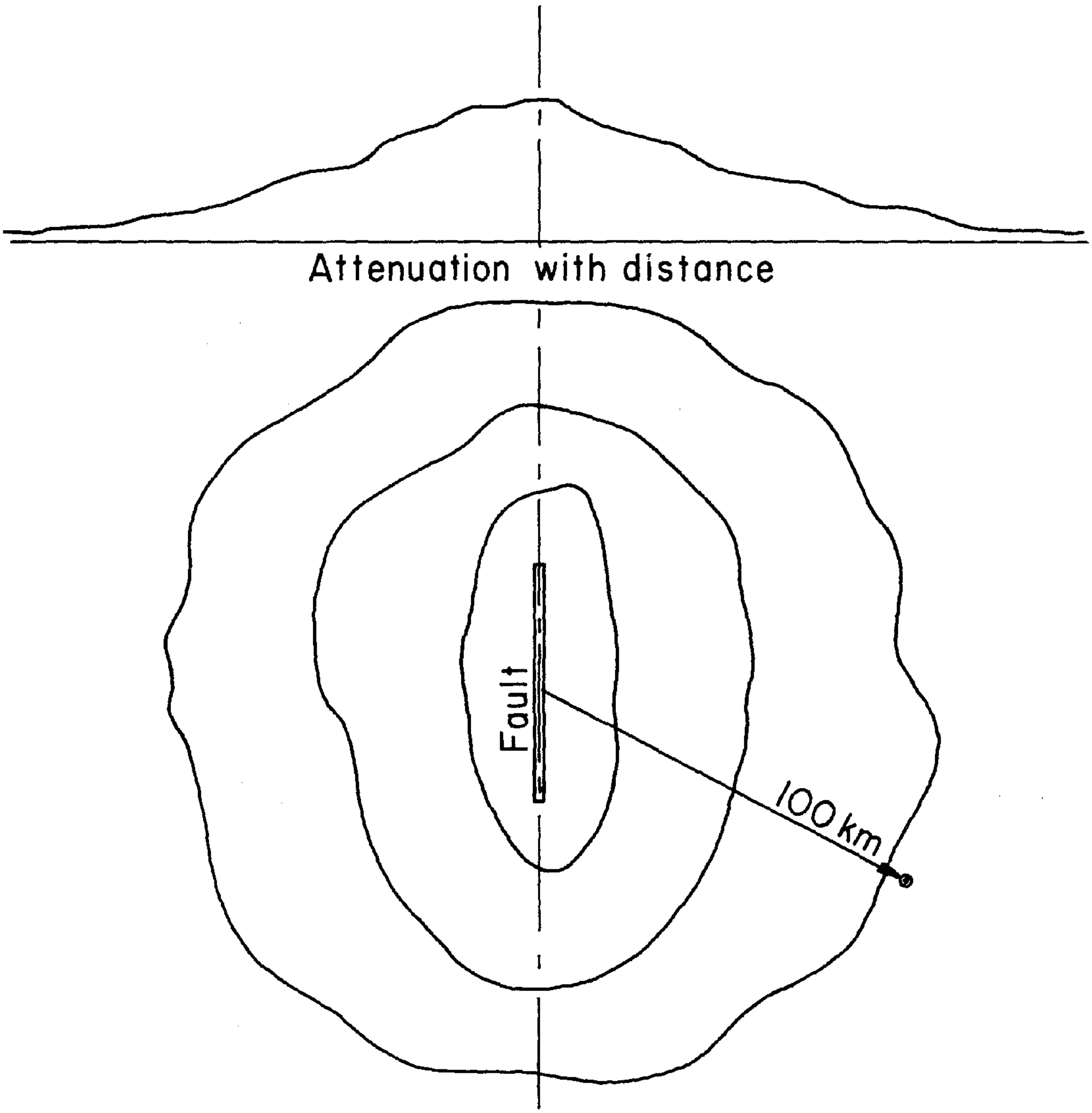


Figure 9. Contour lines of equal values of $\log(A/A_0)$. The Richter magnitude is defined to be $M_L = \log(A^*/A_0)$. A^* is the maximum amplitude of a Wood-Anderson seismograph at 100 kms. A_0 is the amplitude corresponding to $M = 0$.

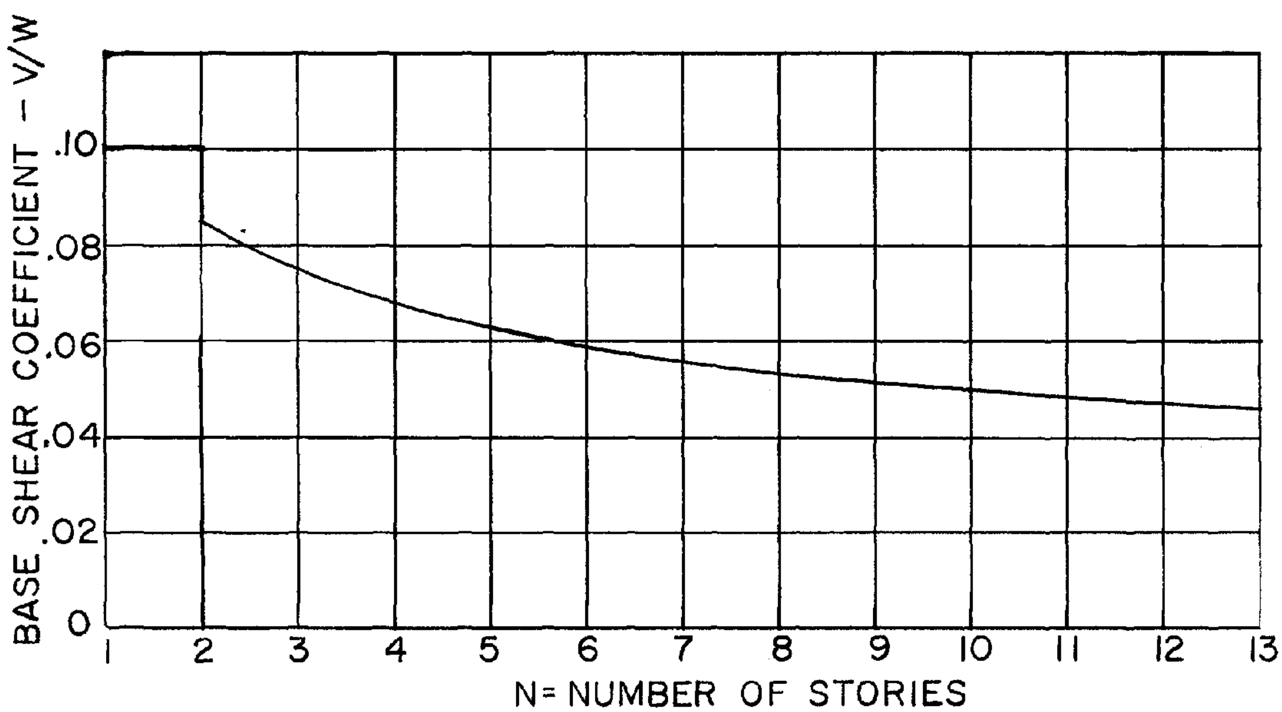
Because of the oblong shape of the contours close to the fault, it is desirable to measure A at a distance that is large compared to the fault length, however at different distances different seismic waves are predominant in the seismogram. For example, within a few miles of the fault the ground motion recorded by a strong motion accelerograph is predominantly short-period shear waves and compression waves from which a magnitude can be computed. At distances of several thousands miles from a large earthquake, surface waves of 20-second period are prominent from which M_s can be computed. At least four different magnitudes are, in practice, computed and they do not, in general, give the same numerical value, though there are techniques for converting from one to another. For example, a strong motion accelerograph close to the center of a $M_s = 7$ earthquake fault (40 miles long) could be expected to record shaking of approximately the same intensity as for a $M_s = 8$ event (200 miles long), other things being equal, so in this case the magnitude based on strong-motion records could not distinguish between $M_s = 7$ and $M_s = 8$. Each different type of magnitude designation loses the ability to distinguish between two sizes of earthquakes at some point in the magnitude scale. Unfortunately, it is not always made clear which magnitude is being used and this can lead to confusion.

For engineering purposes, the magnitude can be taken as an approximate measure of the size of the earthquake; that is, the area affected by strong ground shaking.

8. APPENDIX II

Earthquake Requirements of 1970 Los Angeles Building Code

The earthquake requirements of the 1970 Los Angeles Building Code are indicative of the seismic design criteria used for the newer buildings that experienced the 1971 San Fernando earthquake. The buildings as actually designed may have deviated more or less from the precise requirements of the code. When comparing the base shear coefficients in the code with the spectral acceleration corresponding to the first mode of vibration it should be kept in mind that the base shear coefficient is equal to V/W, that is, the base-shear divided by the total weight whereas for typical multi-story frame buildings the computed first-mode base-shear is $S_a kW$, where S_a is the spectral acceleration and k is approx .75. Therefore, the base shear coefficient times about 1.33 is approximately equal to S_a. The following diagram is a plot of the code base-shear coefficient for ordinary multi-story frame buildings. (K = 1.00)



Box System is a structural system without a complete vertical load-carrying space frame. In this system the required lateral forces are resisted by shear walls as hereinafter defined.

Shear Wall is a wall designed to resist lateral forces parallel to the wall. Braced frames subjected primarily to axial stresses shall be considered as shear walls for the purpose of this definition.

(c) **Symbols and Notations.** The following symbols and notations apply only to the provisions of this Section.

- C = Numerical coefficient for base shear as defined in Section 2305(d)1.
- C_p = Numerical coefficient as defined in Section 2305 (d)2 and as set forth in Table No. 23-B.
- D = The dimension of the building in feet in a direction parallel to the applied force.
- D_s = Plan dimension of the vertical lateral force resisting system in feet.
- F_a = Allowable axial stress.
- f_a = Computed axial stress.
- F_b = Allowable bending stress.
- f_b = Computed bending stress.
- F_i, F_n = Lateral force applied to level i or n respectively.
- F_p = Lateral forces on the part of the structure and in the direction under consideration.
- F_t = That portion of V considered concentrated at the top of the structure, at the level n. The remaining portion of the total base shear V shall be distributed over the height of the structure, including level n.
- F_x = Lateral force applied to a level designated as x.
- h_i, h_n = The height in feet above the base to level i or n respectively.
- h_x = Height in feet above the base to the level designated as x.
- J = Numerical coefficient for base moment as defined in Section 2305(h).
- J_x = Numerical coefficient for overturning moment at level x.
- K = Numerical coefficient as set forth in Table No. 23-C.
- Level i = Level of the structure referred to by the subscript i.
- Level n = That level which is uppermost in the main portion of the structure.
- Level x = That level which is under design consideration.
- M = Overturning moment at the base of the building or structure.
- M_x = The overturning moment at level x.
- N = Total number of stories above exterior grade to level n.
- T = Fundamental period of vibration of the building or structure in seconds in the direction under consideration.
- V = Total lateral load or shear at the base $= (F_t + \sum_{i=1}^{n} F_i)$ where i = 1 designates first level above the base.
- W = Total dead load $= (\sum_{i=1}^{n} w_i)$

  EXCEPTION: W shall be equal to the total dead load plus 25% of the floor live load in storage and warehouse occupancies.
- W_p = The weight of a part or portion of a structure.
- w_i, w_x = That portion of W which is located at or is assigned to level i or x respectively.

(d) **Minimum Earthquake Forces for Structures.** 1. **Total Lateral Force and Distribution of Lateral Force.** Every structure shall be designed and constructed to withstand minimum total lateral seismic forces assumed to act non-concurrently in the direction of each of the main axes of the structure in accordance with the following formula:

$$V = KCW$$

The value of K shall be not less than that exhibited in Table 23-C. The value of C need not exceed 0.10 and shall be determined in accordance with the following formula:

$$C = \frac{0.05}{\sqrt[3]{T}}$$

EXCEPTION: C shall be 0.10 for all one and two story buildings.

T is the fundamental period of vibration of the structure in seconds in the direction under consideration. Properly substantiated technical data for establishing the period T for the contemplated structure may be submitted.

In the absence of such data, the value of T for buildings shall be determined by the following formula:

$$T = \frac{0.05\, h_n}{\sqrt{D}}$$

EXCEPTION: In all buildings in which the lateral force resisting system consists of a moment-resisting space frame which resists 100% of the required lateral forces and which frame is not enclosed by, or adjoined by, more rigid elements which would tend to prevent the frame from resisting lateral forces, the period T shall be computed as follows:

$$T = 0.10\, N$$

The total lateral force V shall be distributed over the height of the structure in the following manner:

A portion F_t of the total lateral force V shall be concentrated at the top of the structure in accordance with the following:

$$F_t = 0.004 \left(\frac{h_n}{D_s}\right)^2 V$$

F_t need not exceed 0:15 V and may be considered as zero for values of $\left(\frac{h_n}{D_s}\right)$ of 3 or less, and

The remainder of the lateral force $(V - F_t)$ shall be distributed over the height of the structure (including the top level) in accordance with the following:

$$F_x = \frac{(V - F_t) w_x h_x}{\left(\sum_{i=1}^{n} w_i h_i\right)}$$

EXCEPTION: One and two-story buildings shall have uniform distribution.

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution on that level.

2. **Lateral force on parts or portions of building or other structures.** Parts or portions of buildings or structures and their anchorage shall be designed for lateral forces in accordance with the following formula:

$$F_p = C_p W_p$$

The values of "C_p" are in Table No. 23-B. The distribution of these forces shall be according to the gravity loads pertaining thereto.

3. **Pile foundations.** Individual pile or caisson footings of every building or structure shall be stayed in all directions at grade level by members capable of transmitting in tension and compression a force equal to 10% of the larger pile cap load. These members may be omitted where it can be demonstrated that an equivalent restraint can be provided by other means.

SEE RULE OF GENERAL APPLICATION #662 IN APPENDIX SECTION

4. **Elevated Tanks.**

A. Designs for elevated tanks on four or more cross-braced columns and not supported by a building shall conform to the following:

The period "T" shall be substantiated by technical data.

The value of "KC" as used in V = KCW in this Subsection shall not be less than 0.12 but need not exceed 0.25.

The factor "J" in the overturning analysis shall be 1.00.

Resistance to horizontal torsion shall be provided and the torsional eccentricity shall be not less than 5% as provided in this Section for buildings.

B. Designs for elevated tanks having arrangements of columns other than in Part A shall use a value of "KC" equal to not less than 0.20 and other provisions of Part A shall apply.

→(e) **Distribution of Horizontal Shear.** Total shear in any horizontal plane shall 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. Rigid elements that are assumed not to be part of the lateral force-resisting system may be incorporated into buildings provided that their effect on the action of the system is considered and provided for in the design. ←

(f) **Drift.** Lateral deflections or drift of a story relative to its adjacent stories shall be considered in accordance with accepted engineering practice.

(g) **Horizontal Torsional Moments.** Provisions shall be made for the increase in shear resulting from horizontal torsion due to an eccentricity between the center of mass and the center of rigidity. Negative torsional shears shall be neglected. In addition, where the vertical resisting elements depend on diaphragm action for shear distribution at any level, the shear resisting elements shall be capable of resisting a torsional moment assumed to be equivalent to the story shear acting with an eccentricity of not less than 5% of the maximum building dimension at that level.

→ (h) **Overturning.** Every building or structure shall be designed to resist the overturning effects caused by the wind forces as set forth in Subsection (l) of this Section, or the earthquake forces specified in this Section, whichever governs.

EXCEPTION: The axial loads from earthquake force on vertical elements and footings in every building or structure may be modified in accordance with the following provisions:

(1) The overturning moment "M" at the base of the building or structure shall be determined in accordance with the following formula:

$$M = J(F_t h_n + \sum_{i=1}^{n} F_i h_i)$$

Where $$J = \frac{0.5}{\sqrt[3]{T^2}}$$

The value of J need not be more than 1.00

(2) The overturning moment "M_x" at any level designated as "x" shall be determined in accordance with the following formula:

$$M_x = J_x [F_t (h_n - h_x) + \sum_{i=x}^{n} F_i (h_i - h_x)]$$

Where $$J_x = J + (1 - J)\left(\frac{h_x}{h_n}\right)^3$$

Seventy-five percent of the dead load may be used to reduce tensile stresses caused by seismic overturning moments.

At any level, the incremental changes of the design overturning moments, in the story under consideration, shall be distributed to the various resisting elements in the same proportion as the distribution of the shears in the resisting system. Where other vertical members are provided which are capable of partially resisting the overturning moments, a redistribution may be made to these members if framing members of sufficient strength and stiffness to transmit the required loads are provided.

Where a vertical resisting element is discontinuous, the overturning moment carried by the lowest story of that element shall be carried down as loads to the foundation.←

(i) **Set-Backs.** Buildings having set-backs wherein the plan dimension of the tower in each direction is at least 75% of the corresponding plan dimension of the lower part may be considered as a uniform building without set-backs for the purpose of determining seismic forces.

For other conditions of set-backs the tower shall be designed as a separate building using the larger of the seismic coefficients at the base of the tower determined by considering the tower as either a separate building for its own height or as part of the overall structure. The resulting total shear from the tower shall be applied at the top of the lower part of the building which shall be otherwise considered separately for its own height.

→(j) **Structural Systems. 1. Design Requirements.** Buildings designed with a horizontal force factor "K" of 0.67 or 0.80 shall have a ductile moment-resisting space frame. Buildings more than 160 feet in height shall have a ductile moment-resisting space frame capable of resisting not less than 25% of the required seismic force for the structure as a whole.

Moment-resisting space frames and ductile moment-resisting space frames may be enclosed by or adjoined by more rigid elements which would tend to prevent the space frame from resisting lateral forces where it can be shown that the action or failure of the more rigid elements will not impair the vertical and lateral load resisting ability of the space frame.

2. **Construction.** The necessary ductility for a ductile moment-resisting space frame shall be provided by a frame of structural steel conforming to ASTM A-7, A-36 or A-441 with moment-resisting connections, or by a reinforced concrete frame complying with Section 91.2670 of this Code.

Shear walls in buildings exceeding one hundred and sixty feet in height shall be composed of axially loaded bracing members of ASTM A-7, A-36 or A-441 structural steel; or reinforced concrete bracing members or walls conforming with the requirements of Section 91.2680 of this Code. ←

(k) **Design Requirements. 1. Combined axial and bending stresses in columns.** Except → for reinforced concrete columns designed in accordance with Division 26 of this Code and except for structural steel columns designed in accordance with Division 27 of this Code, all structural columns shall conform to this subdivision. The maximum allowable extreme fiber stress in columns at intersection of columns with floor beams or girders for combined axial and bending stress shall be the allowable bending stresses for the material used. Within the center one-half of the unsupported length of the column, the combined axial and bending stresses shall be such that:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \text{ is equal to or less than 1.}$$

When stresses are due to a combination of vertical and lateral loads, the allowable unit stresses may be increased as specified in Subsection 91.2301(g). ←

2. **Building Separations.** All portions of structures shall be designed and constructed to act as an integral unit in resisting horizontal forces unless separated structurally by a distance of at least one inch, plus ½ inch for each 10 feet of height above 20 feet.

3. **Minor Alterations.** Minor alterations may be made in existing buildings and other structures, but the resistance to lateral forces shall be not less than that before such alterations were made, unless the building as altered meets the requirements of this section of the Code.

→4. **Reinforced Masonry or Concrete.** All elements within a structure which are of masonry or concrete shall be reinforced so as to qualify as reinforced masonry or concrete under the provisions of Divisions 24 and 26. Principal reinforcement in masonry shall be spaced four feet maximum on center except that a maximum spacing of two feet on center shall be used in buildings utilizing a ductile moment-resisting space frame. ←

5. **Combined Vertical and Horizontal Forces.** In computing the effect of seismic force in combination with vertical loads, gravity load stresses induced in members by dead load plus design live load, except roof live load, shall be considered.

→6. **Minor Rigid Elements.** Minor rigid elements within or attached to a structure may be assumed to be expendable and not part of the lateral force resisting system.

7. **Exterior Elements.** Precast, non-bearing, non-shear wall panels or other elements which are attached to, or enclose the exterior, shall accommodate movements of the structure resulting from lateral forces or temperature changes. The concrete panels or other elements shall be supported by means of poured-in-place concrete or by mechanical fasteners in accordance with the following provisions:

A. Connections and panel joints shall allow for a relative movement between stories of not less than two times story drift caused by wind or seismic forces; or ¼-inch whichever is greater.

B. Connections shall have sufficient ductility and rotation capacity so as to preclude fracture of the concrete or brittle failures at or near welds. Inserts in concrete shall be attached to, or hooked around reinforcing steel, or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

C. Connections to permit movement in the plane of the panel for story drift may be properly designed sliding connections using slotted or oversize holes or may be connections which permit movement by bending of steel. ←

(l) **Wind Pressures.** The amount of wind pressure shall be assumed to be not less than the values exhibited in Table No. 23-D.

TABLE NO. 23-D — ASSUMED WIND PRESSURE

Structure or Portion of Structure	Wind Pressure (lbs. per sq. ft.)
Portions more than 60 ft. above adjacent ground elevation	20
Portions less than 60 ft. above adjacent ground elevation	15
Roof structures more than 60 ft. above adjacent ground elevation	30
Roof structures less than 60 ft. above adjacent ground elevation	20
Greenhouses and lath houses	5

The wind pressure shall be assumed to act inwardly or outwardly in any direction. The projected area of the structure upon a vertical plane shall be used in computing the total horizontal wind pressure. Where vertical uplift may occur as the result of a horizontal wind force, the amount of wind pressure acting in an upward direction shall be assumed to be not less than 15 pounds per square foot.

In computing the effect of wind in combination with vertical loads, all vertical loads except the roof live load shall be considered.

The dead load moment of stability of every structure shall be not less than one and one-half times the overturning moment caused by the application of wind pressure specified in this section. The dead load moment of stability shall exclude the moment induced by special loadings such as the contents of vats, tanks and bins.

EXCEPTIONS: 1. For open frameworks the wind shall be applied to 1½ times the projected area of each face of the structure on a vertical plane normal to the direction of the wind.

2. Two-thirds of the projected area shall be used in computing the wind pressure upon cylindrical structures.

(m) **Unenclosed Buildings.** The wind pressure upon the side of a building shall be computed upon the gross area of the side of the building, including all openings in the exterior wall, and including the vertical projection of the roof.

EXCEPTION: Roof shelters over fueling areas of motor vehicle service stations may have the wind pressure computed upon 1½ times the area of those surfaces exposed to the wind.

SEE RULES OF GENERAL APPLICATION #19-68 AND #29-69 IN APPENDIX SECTION

SEC. 91.2306 — DESIGN FOR HORIZONTAL FORCE

(a) **Truss Bracing.** Roof trusses shall be cross braced in a vertical plane normal to the plane of the trusses.

Cross bracing shall be installed at intervals not greater than 30 feet measured in a direction parallel to the trusses.

Cross bracing shall be installed in every fourth bay between trusses. Continuous struts shall be installed between panels of cross bracing in the plane of the lower chord of the trusses.

Struts and cross bracing shall be designed to withstand a horizontal force equal to 10% of the stress in the lower chord of the roof truss.

EXCEPTION: This Subsection shall not apply to exposed trusses in public assembly rooms.

(b) **Anchorage.** Concrete or masonry walls shall be anchored to all floors and roofs. Such anchorage shall be capable of resisting the horizontal forces specified in Section 91.2305, or an arbitrary minimum force of 200 pounds per linear foot of wall, whichever is larger. Where anchorage is intermittent, anchors shall be spaced not more than four feet apart. For wood floor or roof construction the joists, purlins, or rafters shall be anchored to the walls to fulfill the requirements of this Subsection. Toe nails or nails driven parallel to the grain of wood shall not be considered effective in meeting the requirements of this Section.

EXCEPTIONS: 1. When walls are anchored to continuous footings, anchorage to floors that are within four feet of footings may be omitted.

2. When roofs, including their supporting joists, beams, or purlins, are constructed of metal and are not designed as diaphragms to resist horizontal forces upon the walls, anchors may be spaced at more than four feet on center.

3. Where wood roof or floor sheathing is fastened directly to a continuous wall plate that is anchored to the top of a concrete or masonry wall, the anchorage of the continuous plate to the wall to resist the required forces may be considered as fulfilling the requirements of this Subsection.

4. Where wood roof or floor sheathing is fastened directly to a continuous ledger that is anchored to a concrete or masonry wall, the anchorage of the ledger to the wall to resist the required forces may be considered as fulfilling the requirements of this Subsection provided:

(i) The ledger is not less than four inches in thickness;

(ii) Ledger bolts are located as close to the top as practicable but not more than four inches below the top edge of the ledger and are spaced not more than four feet on center.

(c) **Rod Bracing Systems.** Rod bracing systems used for the purpose of transferring horizontal forces in buildings or structures with masonry or concrete walls shall be adjustable.

All adjustable rod bracing systems shall be maintained as set forth in Section 91.0103 of this Article.

(d) **Shear in Concrete Frames.** All concrete frames subjected to reversals of stresses resulting from wind or seismic forces, → and not designed as a ductile moment-resisting space frame under Division 26 of this Code ← shall be provided with sufficient closed loop stirrups to resist all shearing forces with no allowable shear given for the concrete. This requirement shall not apply to column strips in flat slabs or flat plate floor systems.

(e) **Earthquake Recording Instrumentation.** 1. **General.** Every building over six stories in height with an aggregate floor area of 60,000 square feet or more, and every building over ten stories in height, regardless of the floor area, shall be provided with three approved recording accelerographs.

2. **Location.** The instruments shall be located in the basement, mid-portion and near the top of the building. Each instrument shall be located in an accessible position.

3. **Non-Retroactive.** The requirements of this Subsection shall not apply to existing buildings, but shall only apply to buildings, the permits for which are issued after July 1, 1965.

(a)

(b)

Figure 10a, b. Eight story building before and after the 1960 Agadir, Morocco earthquake. In effect, this building was desi,ned for weak ;round shakin ; and no ductility.

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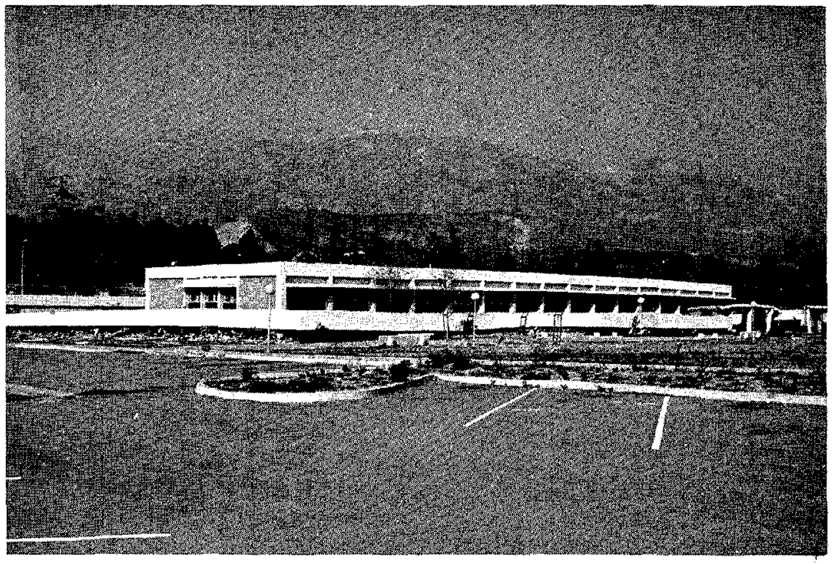
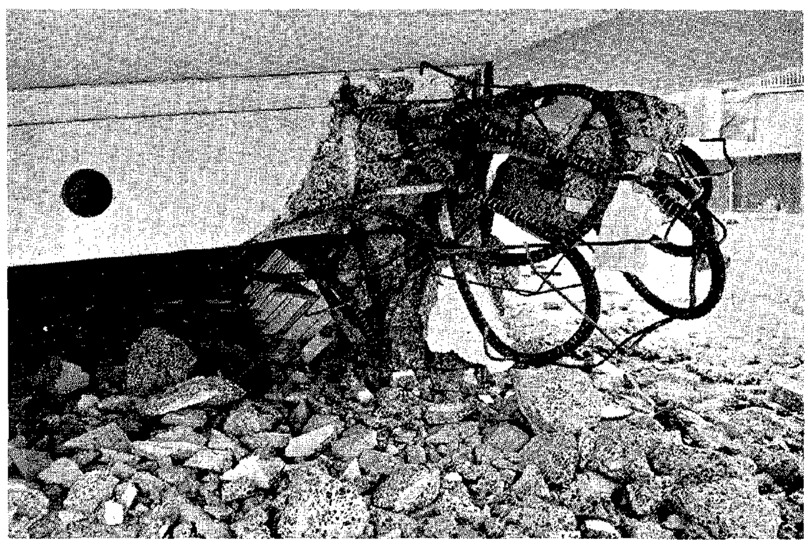
Figure 11. Olive view Hospital after the San Fernando earthquake. The spirally reinforced columns were severely deformed with interfloor displacements of 50 or more times the yieldpoint displacement. Estimated peak ground acceleration was approximately 50%g and the corresponding elastic response spectral acceleration is 1.2g for 0.5 sec period and 5% damping; the corresponding first-mode base shear during the earthquake was governed by the maximum yield moment of the columns.

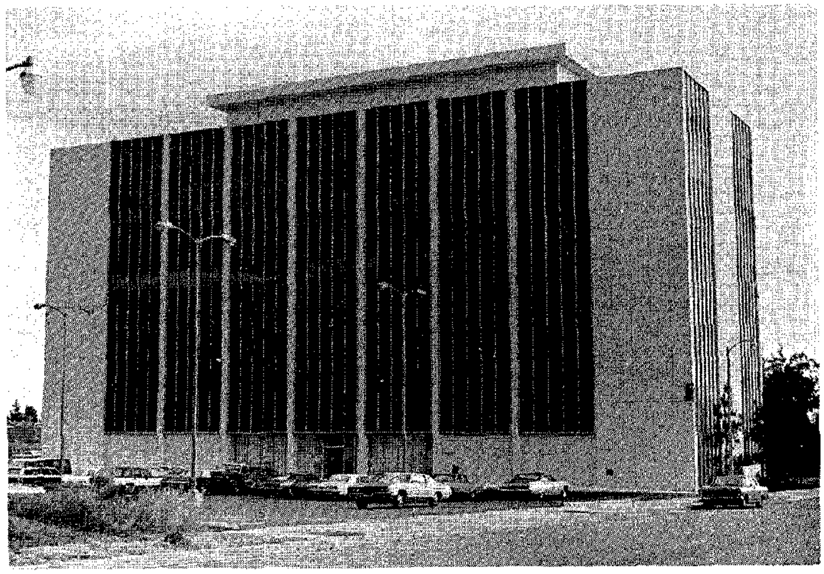


(a)



(b)

Figure 12a,b. Two-story Psychiatric Unit building at Olive View Hospital after the San Fernando earthquake. The The columns of this code designed structure, poorly tied, were made of light-weight concrete which disintegrated under the large strains, as shown in Figure 12b. At 50%g ground motion the spectral acceleration is approx 80%g for .15 period and 5% damping.



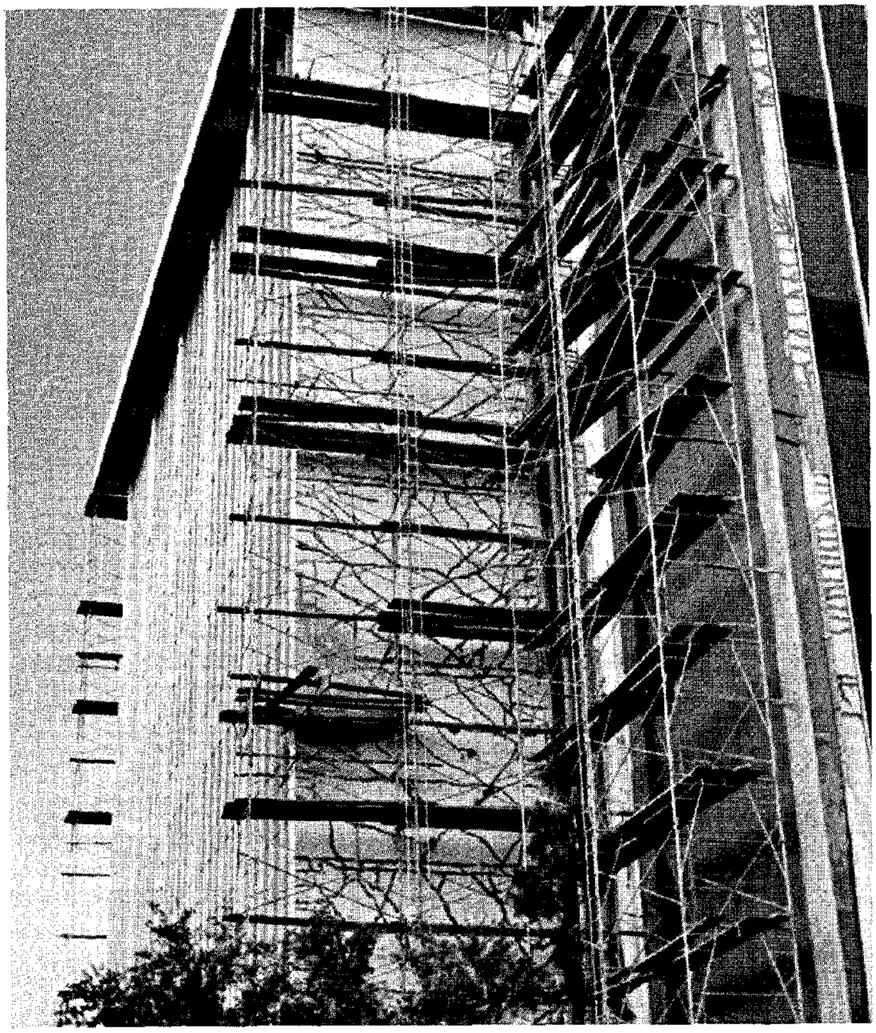


Figure 13a,b. Indian Hills Medical Center building. This 7-story concrete building underwent large strains during the San Fernando earthquake which extensively cracked the vertical cantilever beams that provided the lateral resistance. The cracks, after being painted, are shown in Figure 13b. This code-designed building performed very well under large stresses and strains. The peak ground acceleration is estimated to have been 40%g with spectral acceleration 60%g for 0.7 period and 5% damping. Code value of base shear is 0.055g.

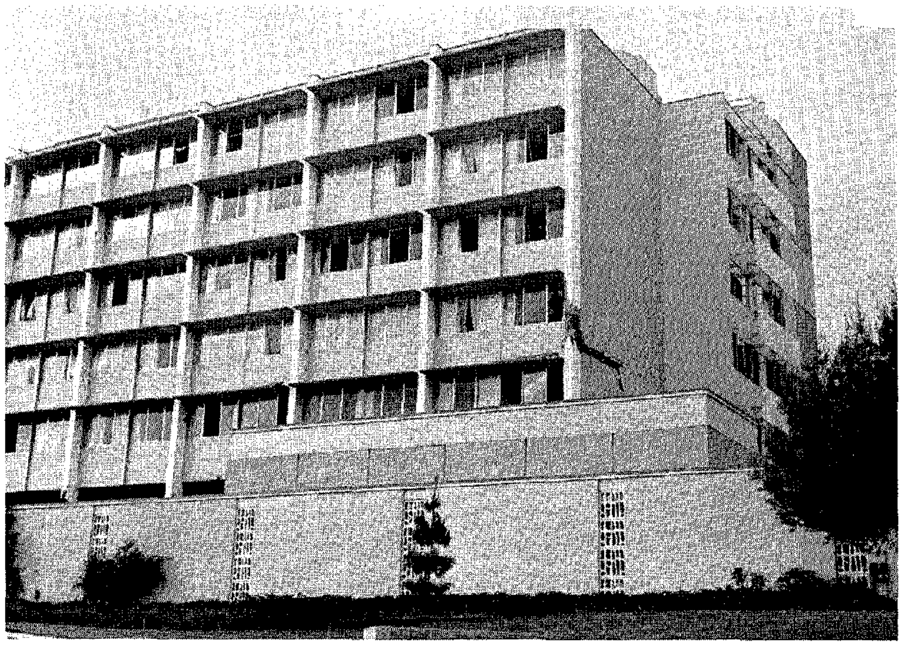


Figure 14. Holy Cross Hospital Building after the San Fernando earthquake. This 7-story concrete frame building suffered severe damage to beams and columns. Estimated peak ground acceleration was 0.4g and the corresponding spectral acceleration was 0.6g for a per--iod of 0.7 sec and 5% damping.

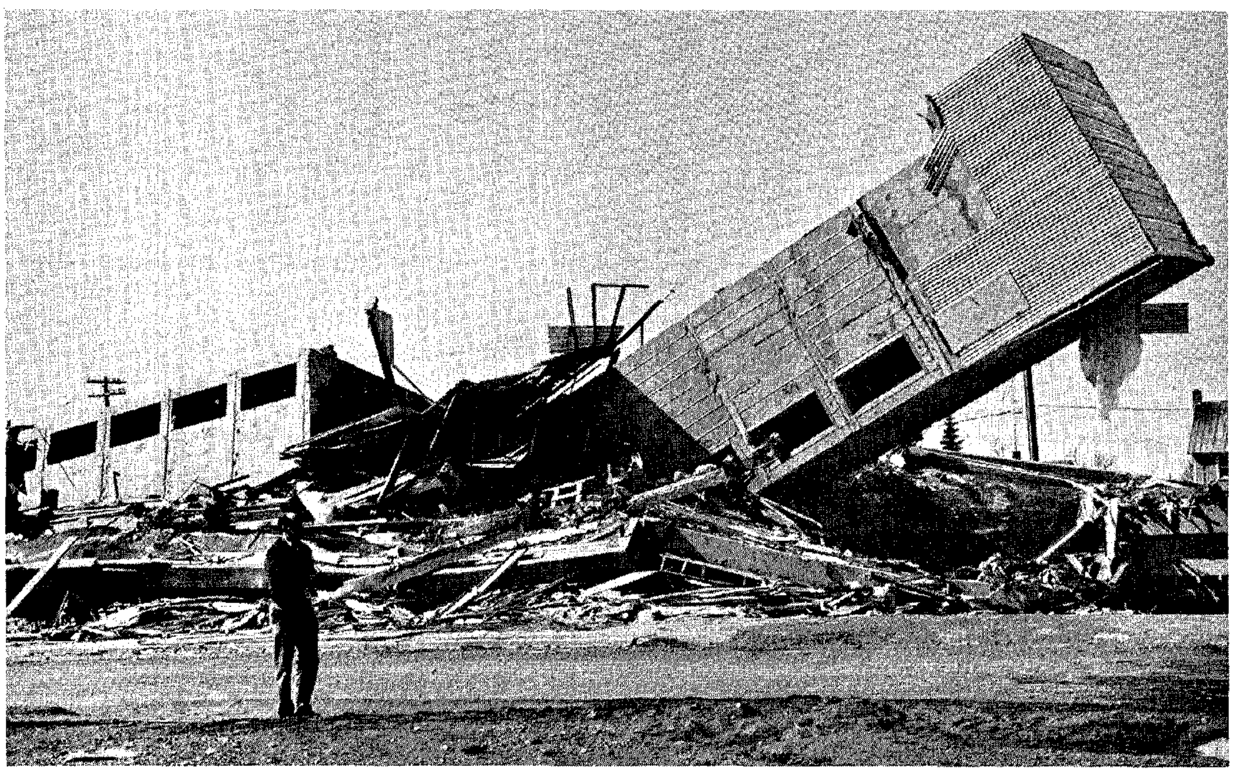


Figure 15. Six story lift-slab Four Seasons Apartment Building in Anchorage following the 1964 Alaska earthquake. It is estimated that the spectral acceleration of the ground motion was approximately 0.4g at .5 secs. and 5% damping. Lateral resistance was provided by the two towers but their connection to the footings was deficient, lapping of bars 20 diameters, and failure occurred at approximately one-half of the bending capacity as determined by the area of the reinforcing bars.

Figure 16. Collapsed freeway overpasses and freeway interchange bridge in Sylmar following the San Fernando earthquake. The estimated peak ground acceleration was approximately 0.5g. The design criteria for these structures were inadequate for such strong ground shaking.

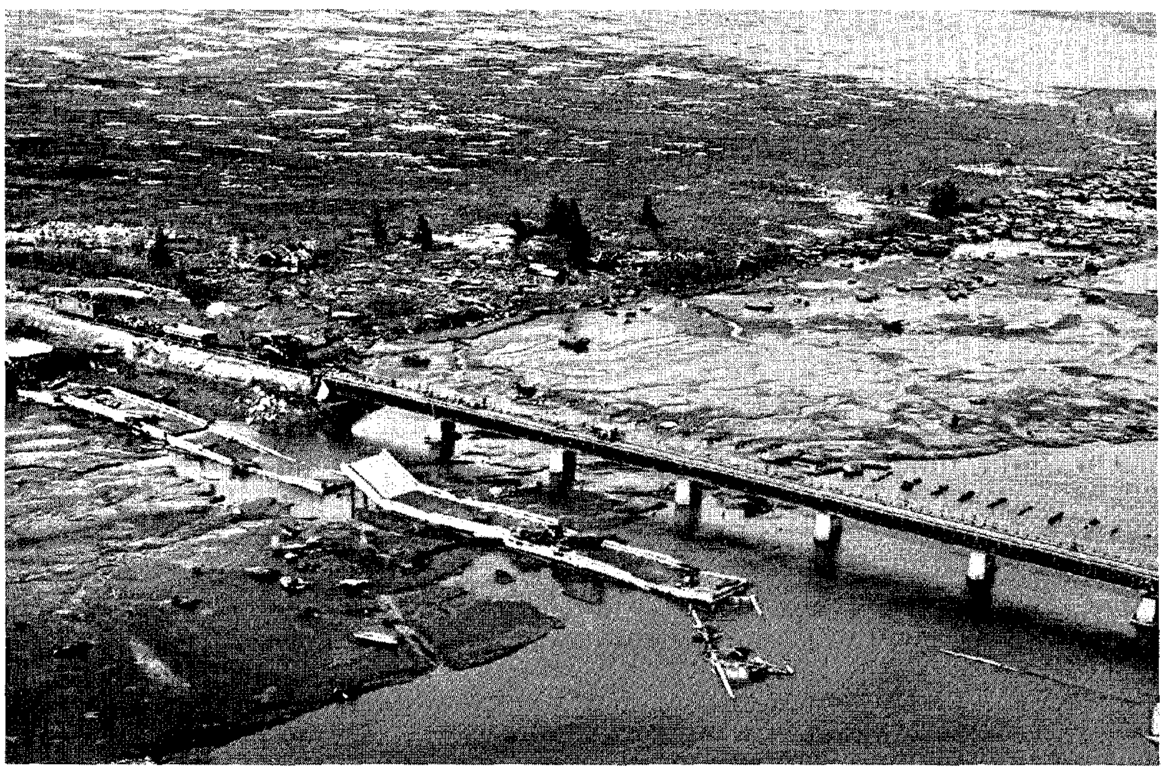


Figure 17. Railroad bridge that survived the 1964 Alaska earthquake and highway bridge that collapsed during the earthquake. The performance of these structures illustrates the effect of adequate and inadequate design criteria.

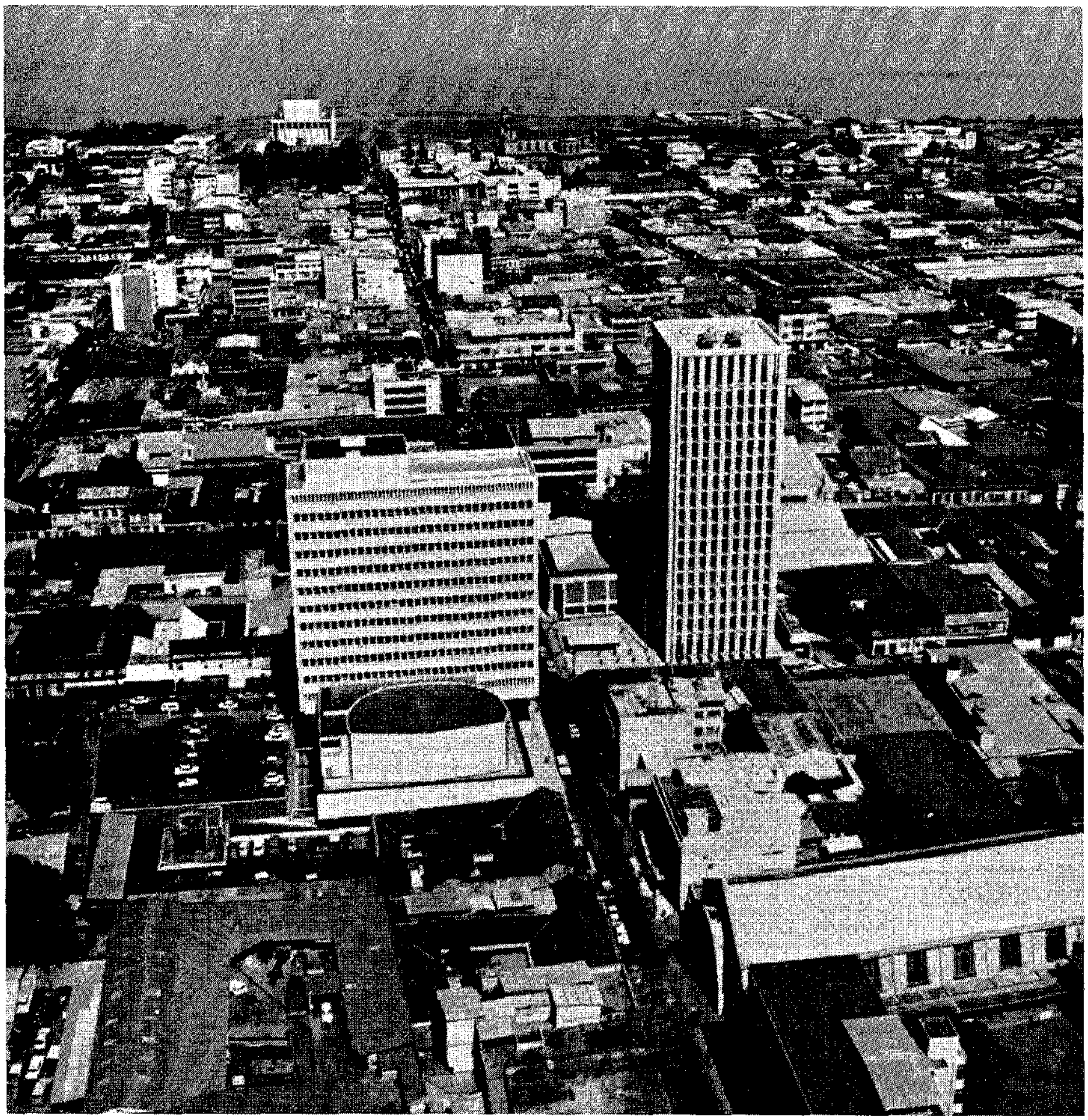
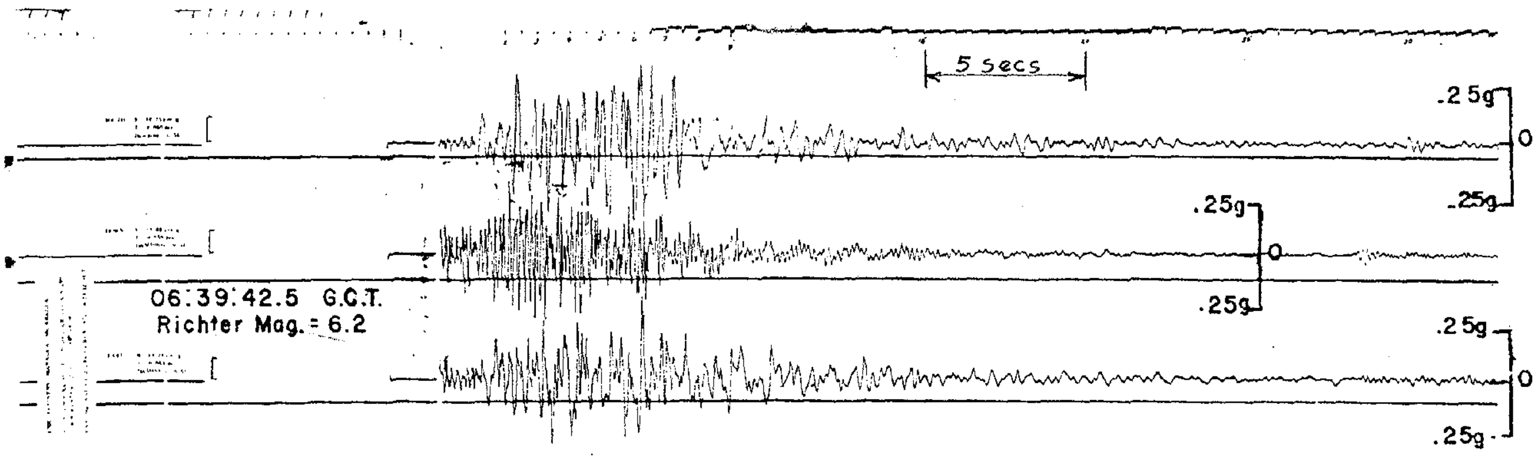
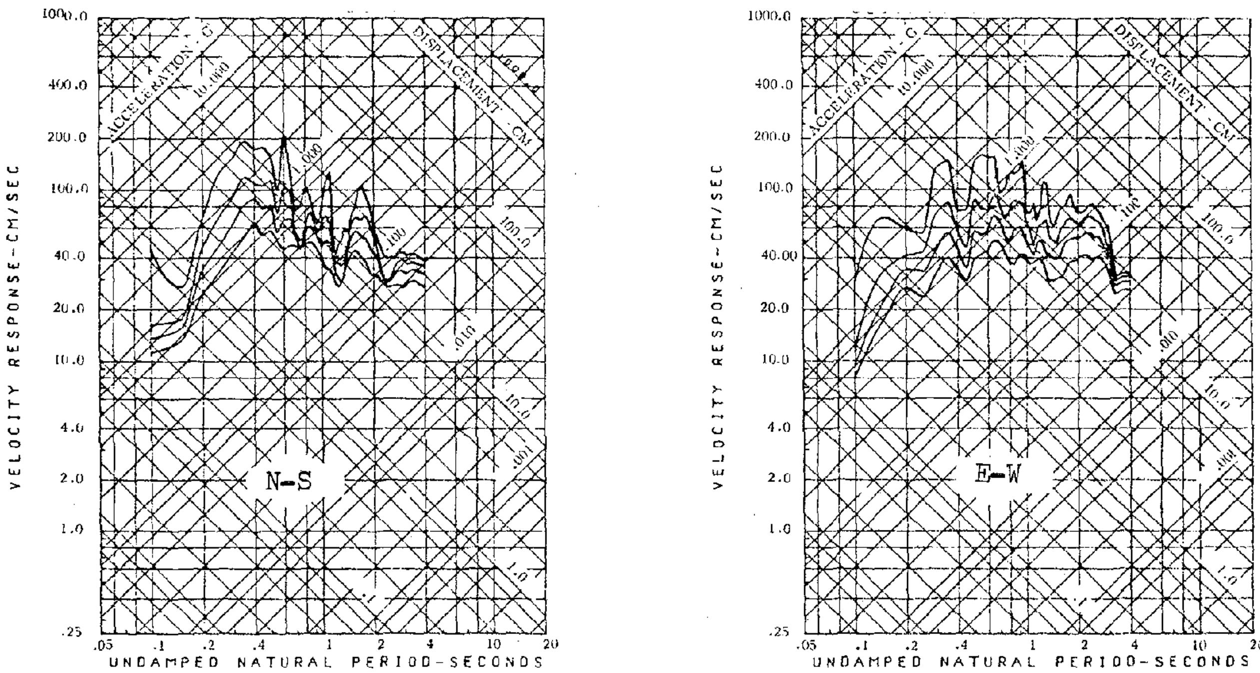


Figure 18. The 18-story concrete shear-wall Banco de America and 15-story Banco Central after the 1972 Managua, Nicaragua earthquake. The shear-wall building sustained moderate structural damage, and the concrete frame 15-story building had little structural damage but rather extensive non-structural damage. both buildings were readily repairable. Both had been designed according to California practise. The peak ground acceleration was approximately 40%g and the corresponding spectral acceleration was 0.17g for 1.8 secs period and 5% damping.



a) Reproduction of original accelerogram (Ref. 3)



b) Response spectra of horizontal components of acceleration (ref. 3). 0, 2, 5, 10% damping.

Figure 19a, b. Ground motion recorded in Managua, Nicaragua during the 23 December 1972 earthquake. The magnitude assigned to this shock (6.2) was somewhat smaller than that assigned to the San Fernando earthquake (6.5). The ground motion was recorded 3 km (2.2 mi) from the surface trace of the causative fault (the two bank buildings were very close to the fault). The duration of the strong horizontal acceleration was 5.5 secs as compared to the duration of 8 secs for the San Fernando earthquake. For periods less than about 1 sec the response spectra of the Managua earthquake are consistent with the San Fernando spectra, but for longer periods the Managua spectra are appreciably lower. Had the Bank buildings been at the site of Indian Hills Medical Center they would have experienced stronger vibrations, approximately $S_a = 0.4g$ instead of 0.17g at 1.8 secs period and 5% damping.