SEISMIC DESIGN CRITERIA FOR NUCLEAR REACTOR FACILITIES

by

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SYNOPSIS

Based on a review of the seismic resistance designs of a number of nuclear reactors in the United States and on independent studies, the authors consider the criteria applicable to the seismic design of nuclear reactor facilities. The criteria described include the selection of the earthquake hazard, ground motions, response spectra, methods of dynamic analysis, methods of designing for fault motion, and a number of special topics. The discussion includes consideration of the relative importance of margins of safety and of applicable stress and deformation levels for facility structures and equipment.

1. INTRODUCTION

The possibility of earthquake damage to a nuclear power facility, as contrasted to a fossil fuel power facility, constitutes a special safety problem because of the possibility of the release of fission products. An uncontrolled release into the atmosphere or cooling waters would constitute a serious hazard to human life in the immediate vicinity of the reactor and could lead to a serious biological hazard over a much larger area for a considerable period of time. The importance of the problem is evident from a recent AEC summary[(1)] that as of March 31, 1968 in the United States there were 82 nuclear power plants either operating, under construction, or planned; an additional 17 plants have been announced. This nuclear electrical power capacity of 69,491 megawatts corresponds to about 26 percent of the total electric utility capacity of the United States by conventional means as of January 30, 1968. Most of this nuclear power capacity has been developed since 1964. Although not as spectacular, significant increases in nuclear power capacity have occurred also in other countries around the world.

In designing nuclear reactor facilities, it has been customary in the U.S. to attempt to classify the structures and equipment into three categories along the lines of those described some years ago by Dr. G. W. Housner.[(2)] The three categories proposed by the authors, and as modified somewhat from those given by Housner, are as follows:

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Class I - Those structures or equipment whose failure clearly might cause or contribute to a nuclear incident, or alternatively those structures or equipment which are believed necessary for containment of fission products and safe shutdown of the plant.

Class II- Structures or equipment essential to the power operation but whose individual failure it is believed would not cause or contribute to a nuclear incident, and would not impair the ability for safe shutdown or containment.

Class III - Structures or equipment convenient to the operation of the facility, but not essential to safety.

Slightly different classifications are in use in other countries but generally the same philosophy has been followed. Since it is impossible in all cases to clearly separate structures and equipment in each of the various categories, it is often necessary to combine and categorically upgrade elements of the facility to ensure that a conservative and safe design will result.

2. SELECTION OF EARTHQUAKE HAZARD FOR EARTHQUAKE RESISTANT DESIGN

Methods currently in use in structural and mechanical design include techniques for the calculation of structural or mechanical response for a time-dependent motion that is accurately described, either in terms of the time history of motion or in terms of some probability intensity coupled with a general pattern of motion. Such calculations lead to computed values of strain or stress in the structure or element, the amount of deformation, and similar quantities. From the results of such calculations, and with the application of judgment and experience, the margin of safety, or the adequacy of the structure or mechanical element, can be estimated for the given earthquake or ground motion hazard adopted in combination with other applicable loadings.

Unfortunately, the earthquake hazard for which the design calculations should be made is not subject to the same degree of certainty or the same type of exercise of judgment as are the methods of analysis. In only a few areas of the world is there a relatively long period of recorded observation of strong motion earthquake intensities. By correlating the data available from strong motion records with that from the more common earthquake records available, and by use of qualitative reports of various effects, some measure can be obtained of the maximum intensity which has occurred, or which may occur in the future, in a specific region. In regions of the world where records are scarce, estimates of a similar nature can be inferred, but are much more uncertain. This earthquake will be designated hereafter as the "operating basis" earthquake.

It is much more difficult to determine the maximum possible intensity of an earthquake that could be expected. The maximum earthquake might never have occurred in the past; certainly it might not have occurred within the period of recorded history. Nevertheless, in order to have a sound basis for structural design of important structures, one must make some estimate of the maximum possible intensity of the earthquake hazard which the structure must survive. This earthquake will be designated herein as the "design basis" earthquake.

The degree of permissible damage, or the closeness to failure of the particular structure or equipment item under consideration under the selected operating basis earthquake hazard, is a matter that requires exercise of judgment in another way. One can adopt the criterion that the design must be optimized for: (1) the additional cost of providing for earthquake resistance for this earthquake loading if it has some assigned probability of occurrence during the lifetime of the structure; and (2) the cost of repairs and loss of power revenues if provision is not made for resistance to this earthquake loading. The degree of additional cost to provide the needed earthquake resistance, and the cost of damage or loss of revenue that might be permissible, can be weighted to enable the designer to make a decision as to the "insurance" cost to provide the necessary strength to resist the operating basis earthquake.

On the other hand, the design basis earthquake might produce severe deformations and even collapse of the structure, and the philosophy of design here is somewhat different. It would not be economical to provide for resistance for such an earthquake with the same factor of safety or margin of safety that one normally uses. Of concern in this case is potential for loss of life or excessive property damage. Hence, the selection of the margin of safety for which the design is to be made for the design basis earthquake is in part dependent on the nature and importance of the structure, its location relative to population centers and the like, and the effects that would be produced should the structure or equipment fail to survive.

Thus the selection of the intensities of the operating basis earthquak and the design basis earthquake is dependent on the region of the country where the site is located, the geologic conditions, and previous seismic experience and records. The way these intensities are used in design is highly dependent on the nature of the structure or equipment, the materials employed, as well as the implications of failure. For example, a much lowe factor of safety might be permissible for a one-story residence or warehous than for a multistory building. An even higher factor might be required fo a dam, the failure of which would flood communities or plants downstream. And an even higher factor of safety might be required for Class I component of a nuclear reactor, where damage might involve exposure of large numbers of people to excessive radiation hazards.

3. EARTHQUAKE GROUND MOTIONS

From a study of a large number of strong motion earthquakes, as well a ground motions arising from other sources such as nuclear blasts, one can estimate the general nature of the structural response of structures and components supported by the ground if one can make reasonably good estimate for the motion history of the maximum ground acceleration, the maximum grou velocity, and the maximum ground displacement. It is also necessary to mak a reasonably good estimate of the total duration of strong motion, or the number of individual large pulses in the displacement, velocity, and acceleration. Although these maxima may vary somewhat with direction or orientation, being possibly somewhat greater in the direction of the fault motions associated with the earthquake, in general the maximum horizontal components of motion do not differ greatly in the various directions, but the vertical components of motion may be somewhat less, or in some circumstances equal to or slightly greater than the horizontal motions, depending upon whether the associated fault motions in the earthquake are primarily horizontal (strike-slip motions) or primarily vertical (thrust fault or dip-slip motions).

In regions such as the West Coast of the United States, the relative intensity of these various motion parameters is usually inferred as proportional to the maximum ground acceleration. However, in other regions this may not be reasonable, and it may be necessary to ascertain independently the controlling maximum ground motions of displacement, velocity and acceleration.

The maximum strong motion record of a major earthquake that exists is that for the El Centro, California earthquake of May 18, 1940. In the North-South horizontal component of motion, the maximum ground acceleration reached during the earthquake was about 0.33g, the maximum ground velocity was about 14 in. per sec., and the maximum ground displacement was 10 to 12 in. These measurements were not taken near a fault. The duration of strong motion was nearly 30 sec. The maximum vertical accelerations were approximately two-thirds the horizontal values.

Although there is a great deal of difference of opinion about the nature of motions in earthquakes in general, it is the authors' opinion from a study of the various earthquake and nuclear shock records that the maximum ground velocity is a much more stable parameter with a more determinate upper bound than the maximum ground acceleration. There are theoretical considerations that lead to the conclusion that the maximum possible ground velocity arising from the strongest possible earthquake cannot exceed a value of the order of about 6 ft. per sec. Estimates have ranged from 3 ft. to 9 ft. per sec. for this upper limit. However, the maximum accelerations might be greater than 1.0g, although the accelerations can be very high for relatively small velocities under certain conditions. However, many areas are certainly less active seismically than such extreme areas, and it would be unreasonable in these areas to assume a maximum credible earthquake of this order of intensity.

In those instances where faults occur under or close to a structure, one must also have some knowledge of the maximum relative fault motions and their direction. It has been fairly well established from observations after major earthquakes, and from the damage or lack of damage to structures near faults, that the maximum accelerations and velocities near a fault are not essentially different from those at some distance away. In other words, there is good reason to believe that the fault motion occurs relatively slowly or at least at a slower rate than the maximum transient motion.

As an approach to the problem where specific information is unavailable, the following is suggested based on a "standard" earthquake somewhat similar to the El Centro earthquake in terms of its ground motions, with the other earthquakes considered being stated in terms of the standard earthquake. The standard earthquake is assumed to have a maximum acceleration of 0.5g, a maximum ground velocity of 24 in. per sec., and a maximum displacement of 18 in. This standard earthquake is approximately 50 percent greater in intensity than the El Centro earthquake. Other earthquakes with ground motion intensities taken as proportional in value to the "standard" earthquake, are described in Table 1.

4. SOIL AMPLIFICATION

The ground motions near the surface of the ground, where a structure might be located, are affected by the properties (stiffness, strength, layering) of the soil and rock strata between that point and the source. In general, the most effective transmission of the earthquake forces and stresses are those corresponding to shear waves transmitted from the underlying rock nearly vertically through the underlying strata to the surface.

The motions in rock, where rock outcrops near the surface, might be very closely related to those in the deep underlying strata, but those in soil can be greatly affected by the phenomena of transmission through the soil. In general, the softer the soil through which the shear waves are propagated, the more the high frequency waves are filtered out, and the greater is the amplification of wave motions that have frequencies corresponding to the natural frequencies of the soil strata structure. For example, in Mexico City, the deep soft clay underlying the major part of the city has a fundamental natural frequency of vibration of about 0.4 cycles per second. Hence, motions in the range of periods from about 2 to 2.5 secs. are much more highly amplified than those in other ranges, and relatively high frequency motions are almost entirely damped out when the motions are intense. Because of the energy absorbing characteristics of soil, and the fact that weak soils may deform in shear at relatively low stresses, the transmission patterns for different strengths of signal may be highly different; the amplifications of high frequencies may be substantial for small input motions, and may be practically negligible for large input motions.

Although some studies have been made of amplification phenomena, the problem cannot be considered as solved and much further work needs to be done. In addition to theoretical studies, in the years ahead ground motion measurements at depth as well as on the surface may lead to significant improvements in our understanding of the behavior involved. Nevertheless, from available data it is known that there can be a major difference in effect of relatively soft soils compared with rock in providing an amplification of motion. This amplification will vary with frequency and intensity of motions and may be as much as a factor of two on the velocities and accelerations. For extremely soft soils, the accelerations may even be decreased, but the displacements and velocities may be increased by at least a factor of two.

5. INTERACTION OF GROUND AND STRUCTURE

In general, a structure founded on rock is subjected to the rock motions at its point of contact, with little ambiguity as to the influence affecting the structural motion. Where the weight of the structure is substantially great compared with that of the rock removed to permit the structure to be built, there may be some feedback from the structure into the rock, modifying the motion slightly, although this is in general relatively small for most structures. However, for structures on soil, particularly for heavy and massive structures, the feedback may be considerable and there is also an influence of energy absorption between the structure and its supporting foundations. This energy absorption can be considered to be of the nature of damping or of a general modification of the input motions, but in any case it involves a reduction in the forces transmitted to a structure founded on soil compared with one founded on rock.

Not so well understood are methods for determining the pressures on the wall of a structure buried to a considerable depth in soil; where intense earthquake motions may be expected, there is reason to believe that there may be an amplification of the lateral soil pressures acting on the side walls and, moreover, depending upon the nature of the structure and geologic conditions, in some cases these loadings may be of a nonuniform nature. The problem becomes more complicated in the case of deeply buried structures which are founded on rock at depth with an overlying soil layer bearing against the walls of the structure, for the upper reaches of soil may undergo somewhat different motion than the base rock.

Pile foundations present another problem. In the case of the response of foundations on long piles, there is evidence to indicate that the piles will deform laterally with the surrounding soil and have little influence on the horizontal motions. Vertical motions, however, may be influenced, depending upon whether the piles point bear on harder strata at depth.

Another factor of great importance is the relative motion between structures, and the effect of ground motions on pipes and other connections between structures.

6. DAMPING AND ENERGY ABSORPTION IN A STRUCTURE

In addition to the energy lost between the structure and its supports on the ground, there are energy losses within the structure itself and at the points of attachment of equipment and components to the structure. The damping in structural elements and components and in supports and foundations of the equipment is a function of the intensity of motion, of the stress levels introduced within the structural component or structure, and of the makeup of the structure and the energy absorption mechanisms within it. The authors have reviewed most of the available literature on damping and recommend use of the values presented in Table 2.

7. DESIGN RESPONSE SPECTRA

The basis and development of response spectra are well documented.[3] Designating the maximum displacement (relative to ground) by the symbol D, the maximum pseudo-velocity (relative to ground) by the symbol V, and the maximum pseudo-acceleration (representing the force in the spring) by the symbol Ag, where g is the acceleration of the gravity, for a simple linear oscillator one has the following relations between these quantities, in terms of ω, the circular frequency of vibration of the oscillator:

$$Ag = \omega V = \omega^2 D$$

With this set of relationships, one can plot on a tripartite logarithmic plot the maximum response quantities for a particular earthquake or other dynamic motion. A response spectrum computed very accurately for the 1940 El Centro earthquake horizontal motions in the North-South direction is shown in Fig. 1 for systems having relative damping of 0, 2, 5, 10 and 20 percent of critical. From figures such as these it can be seen that for high frequency systems, higher than about two cycles per second, the maximum acceleration response is nearly constant, but is a function of the damping in the system and the amplification of the response acceleration as compared to the ground acceleration is quite large for lightly damped systems; for extremely high frequency systems, the maximum response acceleration is the same as the maximum ground acceleration. For relatively low frequency systems, having frequencies less than about 0.3 cycles per second, the maximum displacement response is nearly constant and the maximum displacement shows only a slight amplification over the maximum ground displacement, but for extremely low frequency systems the maximum response displacement is exactly the same as the maximum ground displacement. In the intermediate range, the maximum pseudo-velocity response is nearly constant, indicating a constant energy absorption over the entire range of frequencies from 0.3 to about 3 cycles per second. The amplifications are strongly influenced by damping.

Similar diagrams can be obtained for structures in which energy is absorbed inelastically in the spring.[3][4]

From relationships such as those described and in the foregoing parts of this paper, one can arrive at general design bases or criteria for structural design, if one has only the maximum components of ground motion, especially the maximum values of displacement, velocity, and acceleration, with even a minimum of knowledge about the time history of these motions.

The amplification factors for various geologic conditions, described in Section 4, may be used to modify the ground motion values selected for a particular region, to account for differences in foundation conditions. Relative values of ground motion for different conditions might generally be taken in the proportions: (a) Competent Rock, 0.67; (b) Soft Rock or Firm Sediment, 1.0; (c) Soft Sediment, 1.5

In general, the vertical intensities can be taken as two-thirds of the horizontal, where the fault motions are primarily horizontal, and equal to the horizontal where the fault motions are expected to involve large vertical components.

It is our belief that in no case should a design basis earthquake be considered for the design of a nuclear reactor or associated facility for which the ground motion values are less than one-fifth of those given for the standard earthquake in Table 1, or in other words, an earthquake based on a maximum ground acceleration of at least 0.10g.

The response spectra for these earthquakes can be approximated by using straight line segments in the response spectrum plot, having for the different degrees of damping about the same ratio to the ground motions as the El Centro response spectrum. Smoothed response spectra are then determined from the ground motions specified for the "standard" maximum design earthquake, and for the damping factors selected, in accordance with the following procedure.

a) The maximum values of ground acceleration, velocity, and displacement are sketched on the tripartite logarithmic chart shown in Fig. 2. These plots give a polygonal curve representing the maximum ground motion values only.

b) The response of items or of equipment, components of structures and complete structures is then determined by amplifying the ground motion values to obtain response spectrum values, using the suggested amplification factors for the various degrees of damping, as shown in the table in Fig. 2. Values may be interpolated between those shown in this table by linear interpolation. An amplified response spectra for 2 percent damping, for an earthquake having a maximum ground acceleration of 0.33g, is shown in Fig. 2.

In the delineation of the complete response spectrum, straight line bounds are used. For displacement the bound is parallel to the ground motion displacement in Fig. 2. At very low frequencies, of little practical interest, of less than 0.1cps, the response spectrum bound should approach the ground motion. For velocity, the bound is parallel to the ground motion velocity in Fig. 2. For acceleration, the bound has three parts: (1) a straight line acceleration bound parallel to the ground motion acceleration in Fig. 2, drawn from the frequency f_1 of the intersection of the amplified velocity bound and the amplified acceleration bound, to a frequency of 4 f_1; (2) a straight line drawn from that point to the ground motion acceleration value at a frequency of 10 f_1; (3) beyond a frequency of 10 f_1, the spectrum bound is the same as the maximum ground acceleration line.

The response spectra used should in any case represent the effect of amplified motions caused by the dynamic response, appropriately smoothed, and taking into account adequately the probabilities of occurrence of the peaks and valleys in the true response spectrum; moreover, the spectrum selected should be "broadband", i.e., encompass a broad frequency band. If for some reason an actual response spectrum for a particular earthquake is used, it is not appropriate to select for design, values that are at the lowest points of such a spectrum since the precise values of frequency of particular components are uncertain, and a small error may move the value laterally to a point that may be at a response value considerably differe from the minimum valley values.

8. METHODS OF DYNAMIC ANALYSIS

Standard methods of analysis are available for the calculation of the response of structures and equipment to earthquake motions. In general, any of the standard methods may be used for the analysis and design of Class I elements or components. Where the response spectrum approach is used with a modal analysis procedure, at least 3 modes of response of the structure should be considered except in those cases where it can be shown qualitatively that either the third mode or the second mode produces negligible response. When appropriate, the modal maxima should be combined using the square root of the sum of the squares of the individual modal values.

In determining the maxima for all sources of stress, the effects of vertical earthquake motion should be combined directly and linearly with the effects of horizontal earthquake motions, and both should be combined directly and linearly with sources of stress from dead load, live load, thermal effects, pressures, and other applicable operating conditions or loadings.

In the calculation of the structural response, whether by modal analysis or otherwise, the structure should be so represented by means of an analytical or computational model that reasonable and rational results can be obtained for its behavior.

9. CLASS II STRUCTURES

For the design of Class II items of structures and equipment, it is recommended that the static seismic force for design be determined from a rigorous dynamic analysis for a selected earthquake level such as that employed for Class I structures and equipment, or alternatively by a recognized method such as the U.S. Uniform Building Code, where the Zone 3 values are considered by us to be applicable to a maximum ground acceleration of about 0.32g, for damping of the order of 7 to 10 percent, for a ductility factor of the order of 3 to 6, with major damage resulting but not collapse.

For other values of damping, the seismic forces given for Zone 3 should be multiplied by an appropriate coefficient depending on the damping factor, as follows: 7 to 10 percent damping, 1.0; 5 percent, 1.5; 2 to 3 percent, 2.0; 0.5 to 1.0 percent, 2.5.

For low ductility values the seismic forces should again be multiplied by factors that range from 2 to 5, and where adequate ductility is not available, it is recommended that the UBC procedure not be used.

For other earthquake hazards than those corresponding to Zone 3, it is recommended that the static seismic forces given by the UBC for Zone 3 be multiplied by the ratio of the appropriate maximum ground acceleration in the particular area to the value of 0.32g corresponding to Zone 3.

10. FAULT MOTIONS

The selection of appropriate values of fault motion is a more complex problem. Relative motions of the two sides of a fault of about 20 ft. or more have been recorded in California[5]. Motions across subsidiary faults close to a major fault may range from a few inches to several feet. Even where faults are some distance away, there can be an appreciable transient (and even permanent) relative displacement between points on the earth at a distance apart corresponding to the spacing between supports of a structure, or between adjacent structures. Where there is no other information available, it is suggested that relative motions and/or fault motions be taken into account in design in such a way that calamitous failure will not result in important structures for relative motions of at least 1 to 3 in. anywhere; for relative motions of 4 to 8 in. close to (within 2,000 ft.) of subsidiary fault systems or inactive major faults; for relative motions of 1 to 3 ft. on or over subsidiary faults or inactive major faults; for relative motions of 3 to 6 ft. close to (within 2,000 ft.) of major active faults; and for relative motions of 5 to 20 ft. or more over major active faults.

To cope with fault motions of large relative displacement is an exceedingly difficult problem; however, it is not too difficult to provide for isolation from fault motions of the order of 5 to 6 feet with relatively simple expedients. For smaller motions little is needed except to insure that the entire structure is founded on a firm raft foundation that can absorb the differential displacements of the ground motion without being overstressed appreciably. Possible energy absorbent materials are foamed cement or foamed lightweight concrete where the properties can be carefully controlled. The use of sand rather than an energy absorbing material is not a good expedient because of the high shearing strength of sand under confinement, and the lack of energy absorption characteristics.

11. SPECIAL SEISMIC CRITERIA TOPICS

There are a large number of other special topics which must be considered in evaluating the seismic adequacy of a reactor facility design Although space limitations preclude discussion, examples of such items are listed below.

Other types of Loadings and Distortions -- Effects associated with accident pressure and temperature, tsunamis, wind, tornadoes and hurricanes (including tangential and forward winds, negative pressure drops and missiles), bouyancy, flooding, and explosions and missiles, as well as differential displacement of buildings and piping, are examples of factors in this category.

Foundations, Cuts, Dams, Sea Walls, and Intake Structures -- For such items the possible effects of subsidence, settlement and liquefaction, as well as the possibilities of leakage, overturning, slides, stability and flooding are examples of items requiring consideration.

Structural and Mechanical Design Considerations -- Factors to be considered here include loading combinations, the methods of analysis, permissible stress and deformation limits, welding procedures, reinforcing bar splicing details, containment lining design (fastening, buckling), prestressing systems and anchorage details, as well as design controls to avoid difficulties with brittle fracture, fatigue, and corrosion. Critical piping and supports must be designed for dynamic loading conditions and must include consideration of the actual support system and its motions, as well as the need and effect of snubbers, shock absorbers, valves, etc.; vessels, reactor internals and equipment must receive similar attention.

Cranes and Stacks -- Cranes should be designed to resist earthquake loadings or dislodgement. Stacks should be designed and located such that any failure will not impair the safety of the facility.

Instrumentation and Controls -- Critical control instrumentation, including mechanical and electrical devices, cabinets, drive mechanisms, valves, batteries and battery supports and the like should be designed for dynamic motions (forces, deflections and tilt) associated with the design basis earthquake.

Quality Control and Inspection -- In terms of implementing criteria and design, perhaps the most important aspect is that of careful quality control and inspection procedures. Continued inspection of certain critical elements following construction is also highly recommended. In a nuclear power plant where safety is critical and the functioning of items so interrelated, the importance of these aspects of the design and construction procedure cannot be overemphasized.

12. ACKNOWLEDGMENT

This paper is based in part on our experience with reviews of the seismic designs of nuclear power plants in the United States for the DRL of the AEC. However, the views presented are those of the authors and are not to be considered as representing the views or policies of the AEC.

13. REFERENCES

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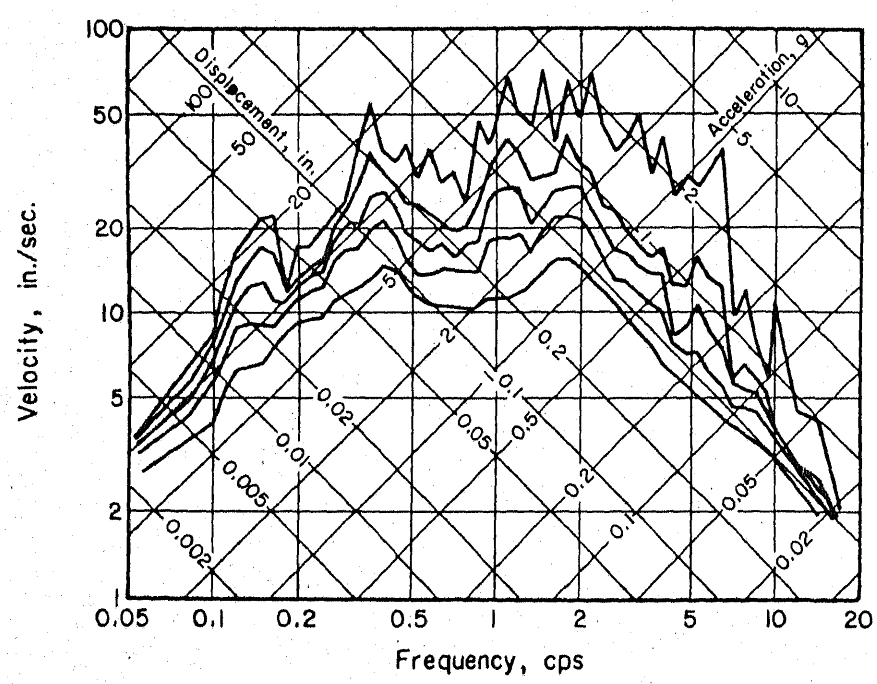
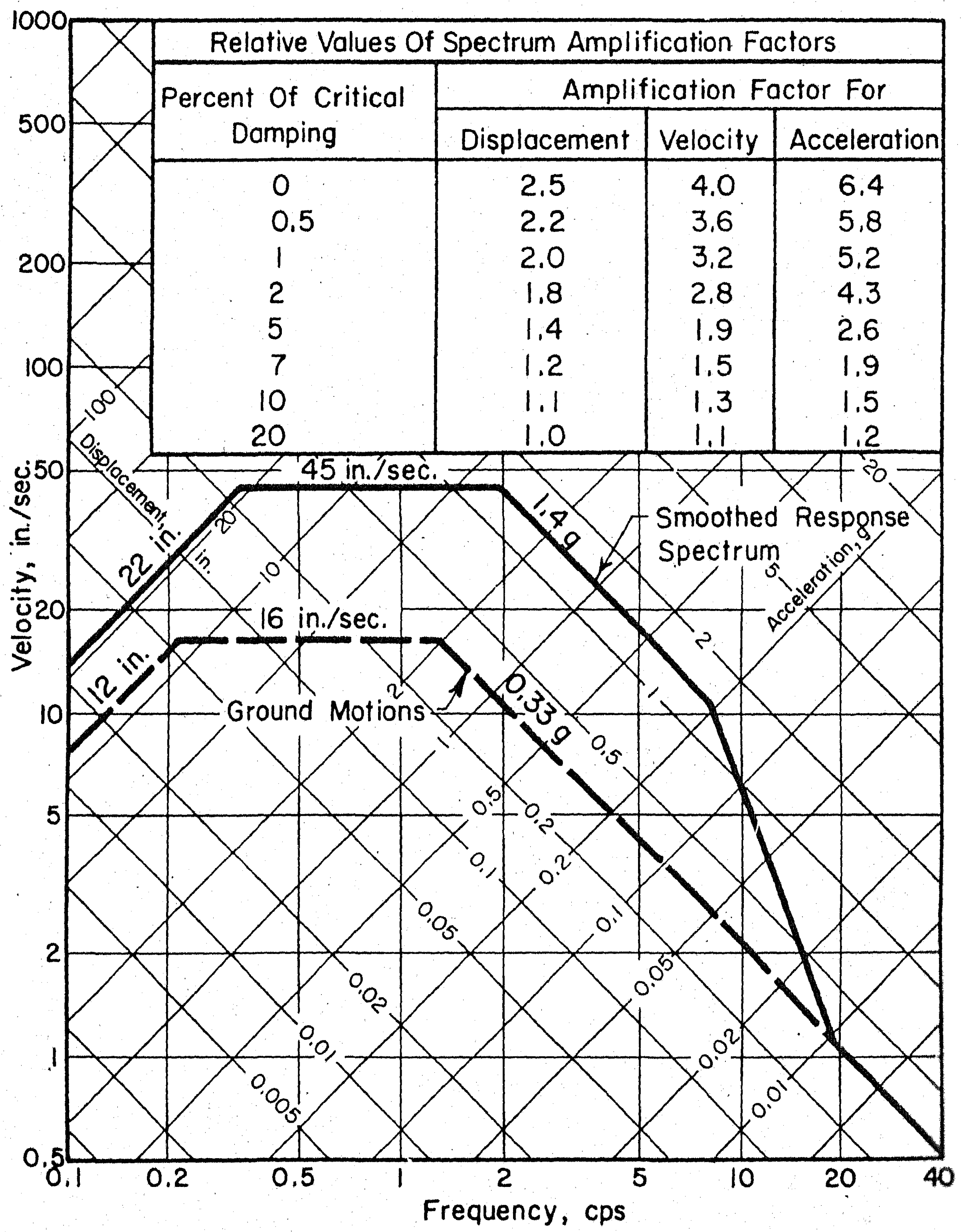


FIG. 1 RESPONSE SPECTRUM, EL CENTRO EARTHQUAKE, MAY 18, 1940, NORTH-SOUTH DIRECTION FOR 0, 2, 5, 10, 20 PERCENT OF CRITICAL DAMPING



Relative Values Of Spectrum Amplification Factors			
Percent Of Critical Damping	Amplification Factor For: Displacement	Velocity	Acceleration
0	2.5	4.0	6.4
0.5	2.2	3.6	5.8
1	2.0	3.2	5.2
2	1.8	2.8	4.3
5	1.4	1.9	2.6
7	1.2	1.5	1.9
10	1.1	1.3	1.5
20	1.0	1.1	1.2

FIG. 2 SMOOTHED TRIPARTITE LOGARITHMIC RESPONSE SPECTRUM FOR 0.33 g EARTHQUAKE, 2 PERCENT CRITICAL DAMPING

TABLE 1

RELATIVE VALUES OF MAXIMUM GROUND ACCELERATION, VELOCITY AND DISPLACEMENT

Condition	Maximum Values of Ground Motion		
	Acceleration g	Velocity in/sec	Displacement* in.
"Standard" Relative Values	0.5	24	18
Typical Maxima			
El Centro, 1940, Horizontal	0.33	16	12
El Centro, 1940, Vertical	0.22	11	8
**Minimum, Horizontal	0.10	5	4
**Minimum, Vertical	0.07	3	3
Very Intense Earthquake	0.75	36	27

* Transient motion <u>not</u> involving relative fault displacement.

** Minimum values recommended for use in design of nuclear reactors in any region, even where earthquakes are not considered probable.

TABLE 2

DAMPING VALUES

Stress Level	Type and Condition of Structure	Percentage of Critical Damping
1. Low, well below proportional limit, stresses below ¼ yield point	a. Vital piping	0.5
	b. Steel, reinf. or prestr. concr., wood; no cracking, no joint slip	0.5 to 1.0
2. Working stress, no more than about ½ yield point	a. Vital piping	0.5 to 1.0
	b. Welded steel, prestr. concr., well reinf. concr. (only slight cracking)	2
	c. Reinf. concr. with considerable cracking	3 to 5
	d. Bolted and/or riveted steel, wood structs with nailed or bolted joints	5 to 7
3. At or just below yield point	a. Vital piping	2
	b. Welded steel, prestr. concr. (without complete loss in prestress)	5
	c. Reinf. concr. and prestr. concr.	7 to 10
	d. Bolted and/or riveted steel, wood structs, with bolted joints	10 to 15
	e. Wood structs with nailed joints	15 to 20
4. Beyond yield point, with permanent strain greater than yield point limit strain	a. Piping	5
	b. Welded steel	7 to 10
	c. Prestr. concr., reinf. concr.	10 to 15
	d. Bolted and/or riveted steel, or wood structs	20
5. All ranges	Rocking of Entire Structure*	
	a. On rock, c > 6000 fps	2 to 5
	b. On firm soil, c ≥ 2000 fps	5 to 7
	c. On soft soil, c < 2000 fps	7 to 10

* Higher damping values for lower values of seismic velocity, c.