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SEISMIC DESIGN CRITERIA FOR NUCLEAR POWER REACTORS

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American Society of Civil
Engineers
Annual Meeting
San Francisco, California
October 7-11, 1963

Based on a Paper Presented at the
ASCE Annual Meeting and
Structural Engineering Conference

San Francisco, California
October 8, 1963

HOLMES & NARVER INC

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SYNOPSIS

The nature of nuclear power reactors demands an exceptionally high degree of seismic integrity. Considerations involved in defining earthquake resistance requirements are discussed. Examples of seismic design criteria and applications of the spectrum technique are described.

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SEISMIC DESIGN CRITERIA FOR NUCLEAR POWER REACTORS

INTRODUCTION

This paper considers factors involved in defining earthquake resistance requirements for nuclear power reactors. Included are examples of seismic design criteria and dynamic analysis of structures under seismic loading. The views presented do not necessarily reflect those of any regulatory agencies.

Before focusing attention on the question of designing reactor structures to resist earthquakes, it might be of interest to digress briefly on the general question of reactor safety and to outline the extent to which seismic considerations are involved. Naturally, a prime interest of every reactor designer is to achieve the safest plant possible. In so doing he must carefully consider that aspect of safety peculiar to reactors: The need for preventing the inadvertent release of the highly radioactive byproducts of the fission process incident to the nuclear chain reaction. To achieve a minimum risk the reactor designer provides several lines of defense against the possibility of releasing radioactivity. These lines of defense, starting from the fuel element and ending with the natural protection provided by site exclusion distances, include primary containment provided by the reactor vessel and other structural materials and secondary containment provided by a gas-tight vapor container designed to resist the overpressures associated with that accident analyzed as the maximum credible accident. Most often the maximum credible accident takes the form of the failure of the primary coolant system and the consequent melting of portions of the fuel. Such melting could be accompanied by the release of fission products from the primary system. In selecting and sizing the lines of defense the intensity of process-generated disruptive forces is generally considered to be controlling. The earthquake problem imposes a distinctly different consideration in that the forces attempting to break through these defenses (e.g., the integrity of the coolant system) are largely unrelated to reactor behavior; that is, they derive from a completely independent external source.

Within this concept of reactor safety requirements, it is of interest to consider past experience in the design and construction of power reactors. In particular, it may be puzzling to some as to why, with the amount of experience already in existence, earthquake resistance requirements are assuming such prominence in reactor safety design. The principal reason derives from the fact that the growth of the nuclear power industry now involves the siting of large power reactors in highly seismic zones, with a correspondingly large increase in the size of the safety problem.

The kind of attention now being focused on the safety issue created by earthquake has, in the past, been directed to the various other issues associated with reactor safety and has been largely responsible for the present favorable experience record of power reactors. In particular, there has been no instance of any radiation injury to any worker in a commercial atomic power plant.

It is a fact of life that there are at present no explicitly defined standards governing the seismic design of reactors in the United States. This paper does not attempt to formulate any such rules but instead is aimed at a presentation of what has been used in the past, based on the writer's own experience, and what has been proposed.

CRITICAL REACTOR SYSTEMS

It is perhaps not a gross exaggeration to say that nuclear reactor structures comprise the only structures wherein it is economically acceptable to design for damage-free response to an extremely severe earthquake. However, even here, the economic yardstick is generally applied with the result that often a design for elastic response is performed only where damage could lead to uncontrolled off-site release of fission products. This paper deals primarily with structures in this category.

Other less critical structures of the reactor may be designed to permit a limited amount of damage under the design earthquake. The non-critical elements, including conventional structures such as warehouses, office buildings, and other appurtenant structures, are often designed for the ordinary building code seismic requirements, which infers the probability, in some instances, of a moderate degree of damage in the design earthquake.

Economic considerations create the need for identifying the components whose malfunction could cause an off-site radiation hazard. For any given reactor, extensive study involving consideration of the various systems involved is necessary to identify all the critical elements in detail. Generally speaking, the critical components include features of the reactor, primary and emergency coolant systems, and containment systems such as the following, along with associated supports, controls, instrumentation and circuitry (Fig. 1):

1. The reactor internals, including the core support structure, fuel elements, control rods and entire rod drive system, including piping and appurtenances.

2. Reactor coolant pumps and prime movers, piping and appurtenances, seals and penetrations at the walls of the pressure vessel and containment vessels.

3. "Poison" injection system (a means of stopping the fission process by injecting neutron absorbing material).

4. Structures identified with containment, such as the reactor pressure vessel, and other surrounding containment structures, including the reactor building.

5. Biological and thermal shields.

6. Fuel storage pool.

7. Control room.

8. Primary and emergency coolant sources, power sources, and fuel sources, including elevated tanks and those on ground; reservoirs; substations, transformers, and systems for emergency power generation.

9. Ventilation systems, including exhaust stacks.

In the case of power reactors, as contrasted to production reactors or test reactors, there is an incentive to stay "on the line" during and after a severe earthquake. Because of this, other components associated with power generation may be placed in the above category.

It should be realized that the important consideration is seismically induced malfunction and that this term is not necessarily synonymous with the term structural failure or inelastic behavior, but covers all effects of an earthquake, including such items as excessive deflection, transient or residual, mismatch of mating elements due to shifting of component parts, short circuits in critical electrical components and similar difficulties. Most of these problems arise in connection with equipment items. With regard to most features whose principal purpose is to house or support critical equipment, the design against malfunction is primarily a stress problem and, occasionally a deflection problem. In the case of critical process components (including piping) similar problems occur and, because of the nuclear threat, attain an importance which requires that their dynamic behavior be considered.

A further area of investigation must consider the possibility of malfunction of critical systems induced by failure of related non-critical systems. It is beyond the scope of this paper to treat the functional analysis of systems or to delve into the complexities of systems analysis. These items are mentioned merely to underscore the fact that they must be properly considered to attain seismic integrity.

FACTORS INVOLVED IN EARTHQUAKE PROTECTION

In any facility the justifiable amount of seismic protection depends principally on three factors:

1. The public importance of the facility in terms of the consequences of earthquake damage.
2. The seismicity of the area, particularly its seismic history with regard to the frequency and intensity of damaging ground motion.

3. The economics of providing protection.

Implementing of protective measures involves certain definite steps to be taken previous to final design. These include:

1. A hazard assessment.

2. A seismicity evaluation.

3. Development of the design basis, which includes identifying critical elements, specification of seismic forces, allowable stresses, assumed percentages of critical damping, and analysis approaches.

The hazard assessment attempts to define the potential consequences of the release of fission products and is also concerned to a lesser degree with other conventional hazards. Considerations involved in the nuclear hazard are:

1. The local population density and distribution.

2. The meteorology of the area, particularly with regard to the prevailing wind direction and velocity and the frequency and extent of atmospheric inversion.

3. Fission product inventory which could result from a credible malfunction of the reactor.

4. The amount, integrity and type of containment relied upon to reduce the off-site hazard to acceptable levels.

The seismicity evaluation attempts to assess the seismic history of the region and from this tries to forecast the maximum probable intensity of the ground motion to be expected. Readily available data for beginning this effort consists of the Uniform Building Code map (see Ref. 1) and the seismic regionalization maps of Richter (Ref. 2). In highly seismic areas the evaluation is usually extended to include the estimates of seismologists, earthquake engineers and geologists.

An assumption very often made is that the maximum intensity of future earthquakes will be no greater than the intensity of past earthquakes. This assumption may not be valid in view of the fact that the records of earthquakes extend at most over a period of not more than several hundred years, which is a mere fraction of a second on a geologic time scale.

Equally important, along with the broad aspects of seismology, is the effect of local geology and foundation conditions at the site, particularly in regard to amplification of base rock intensity by the overlying alluvium, and with regard to an assessment of the possibility of gross soil movement, such as that resulting from slippages of faults underlying the site, large differential movements at fissures in underlying rock formations or gross consolidation of the soil or landslides. It would generally be impossible to design a reactor facility to survive the large displacements associated with these phenomena.

In assessing the earthquake threat at a given site, seismologists and earthquake engineers sometimes attempt to evaluate the so-called maximum probable earthquake. For purposes of this paper, the maximum probable earthquake is defined as an earthquake of that maximum intensity considered to have a reasonable chance of occurrence at the site. Obviously the term "reasonable" in the foregoing definition is vague but is about as definite a statement as can be made regarding a statistical phenomenon evaluated on the basis of varying seismological opinion.

In approaching the earthquake problem associated with siting a large reactor in a highly seismic area, those involved in the public safety aspects of reactor design and operation tend to favor as a design basis the use of an earthquake intensity higher than that of the maximum probable earthquake, but still having a finite chance of occurrence. For purposes of this paper, this earthquake will be termed the "maximum" earthquake.

From the standpoint of public safety the question to be answered is "What is the earthquake intensity beyond which the integrity of the reactor would be severely threatened?" In the writer's opinion the most acceptable estimate of this limit is obtained, in the case of enclosing and supporting structures, when the design is based on an "overload" concept wherein the maximum earthquake produces a condition of impending malfunction. Where such a malfunction is a stress problem, the use of allowable stresses at or near the yield point is often appropriate, and where malfunction is a deflection problem, computed deflections can usually be at the upper limit of tolerance.

In applying this approach to the components of process systems, it can be argued that, to achieve a consistency in true reserve capacity, a higher earthquake input is required than that used for enclosing and supporting structures. The proposed Japanese seismic design criteria for reactors is apparently based on this line of reasoning.

QUANTITATIVELY DEFINING THE EARTHQUAKE

With results of the hazards assessment and seismicity evaluation at hand, there comes the problem of quantitatively defining the earthquake-induced forces — a process which is always argumentative, often arbitrary, and dependent on engineering judgement. This can be done in several ways. One way consists of specifying a static lateral force coefficient. Such coefficients may be based on building code values or some multiple thereof, or may be based on the considered judgement of seismologists or other authorities as to the intensity of the ground motion, often expressed in terms of Mercalli intensity or in terms of a maximum ground acceleration. Expression as Mercalli intensity is less satisfactory for engineering purposes than ground acceleration because of the uncertainties in relating Mercalli intensity to the pertinent ground motion parameters.

Since the seismic effect on a structure depends on the vibrational characteristics of the structure as well as ground acceleration and other properties of the earthquake motion, the maximum ground acceleration used as a lateral force coefficient may not be adequate for design of all critical reactor structures without applying modifying factors to allow for amplification of seismic effects due to the oscillation of the structure.

In the United States the availability of strong motion earthquake records has stimulated an approach involving the use of earthquake spectra. By way of brief explanation, this technique considers the earthquake response of a hypothetical single mass oscillator with one degree of freedom and a specified amount of damping. The spectrum usually is a plot of the maximum elastic response of the oscillator to a given earthquake against various assumed values for the natural period of the oscillator. By maximum response is meant its maximum acceleration, its velocity relative to the ground, or its maximum displacement relative to the ground. Figure 3 shows velocity spectra for the N-S component of the 1940 El Centro earthquake. As shown, the spectra are highly irregular, especially when the damping is small. For design purposes, conventionalized spectra consisting of smooth curves have been proposed (Refs. 3, 4 and 5).

Figure 4 represents the conventionalized velocity spectra of Dr. Housner, obtained by averaging normalized velocity spectra of the N-S and E-W components of four of the strong motion earthquakes in the western United States for which instrumental data is available. The scale of these curves has been adjusted to conform to the intensity of the ground motion in the El Centro earthquake of 1940. This ground motion is the most intense for which instrumental data is available. The spectra of Fig. 4 are sometimes considered to represent the maximum probable earthquake for the zone 3 areas of the Uniform Building Code map of Fig. 2 (Ref. 34); that is, for areas where major destructive earthquakes have occurred in the past and might reasonably be expected at any future time. This area includes a major part of California and Nevada and certain other more localized areas throughout the United States. Figure 5 recasts the spectra of Fig. 4 in a four-way logarithmic plot which is useful in eliminating a certain amount of calculation needed in design applications of Fig. 4. It also avoids the need for a special detailed plot in the short-period range.

Newmark (Ref. 5) has proposed an upper bound spectrum for structures having damping in the range of 5 to 10% critical which is useful where earthquake records are lacking. This spectrum is synthesized from predictions of the maximum ground acceleration, velocity and displacement, and is best represented on a four-way logarithmic plot such as that in Fig. 5. It consists of three lines as follows:

1. A sloping acceleration line plotted at twice the maximum ground acceleration.

2. A horizontal velocity line plotted at 1.5 times the maximum ground velocity.

3. A displacement line having a displacement equal to the maximum ground displacement.

An upper bound spectrum of this type is plotted in Fig. 5 (dotted lines) for the following parameters obtained for the N-S component of the 1940 El Centro earthquake as given in Ref. 5.

Acceleration	33%g
Velocity	1.14 ft/sec
Displacement	0.69 ft

It is seen that this spectrum bounds the average spectrum curve for 5% damping over essentially the entire range of the chart.

In reactor design, conventionalized spectra have been used to represent the maximum earthquake by scaling the ordinates of Fig. 4 to fit the assumed maximum ground acceleration being considered. The scale factor can be greater or less than unity, depending on whether the assumed ground acceleration is greater or less than the 33%g maximum ground acceleration of the 1940 El Centro earthquake. In recent years such spectra have been used in the development of seismic criteria for the design and review of certain reactor facilities in the western United States. The choice of the scale factor can be highly controversial and, in the writer's experience, it usually is. The "scatter band" of authoritative opinion regarding the maximum acceleration of the maximum earthquake in California appears to range from 50%g (Ref. 6) to 100%g or more, implying a scale factor of 1.5 to 3 or more applied to the ground motion of the El Centro 1940 intensity.

SPECTRA AND RELATED PARAMETERS

At this point it should be emphasized that the use of a given set of spectra does not, in itself, define the required earthquake resistance to be built into the structure. The stress levels to be used in design, percentages of critical damping to be used, and the design approach are also involved.

As previously indicated, when the maximum earthquake is used as a design basis, it is often considered appropriate to use elevated stresses, approaching the yield point in some cases. For example, the basic normal working stresses for reinforced concrete might be increased by factors of from 1.5 to 2, the higher figure being used where it can be assured that adequate reserve strength in shear and bond can be maintained. Basic working stresses for structural steel might be increased by a factor as large as 1.6. On the other hand, if a working stress basis is used, it is apparent that a structure of essentially equivalent strength will result if the ordinates of the design spectra are correspondingly reduced. Thus, if the elevated stresses used with the spectra of the maximum earthquake are twice the normal working stresses, spectra having ordinates half those of the maximum earthquake spectra could be used in conjunction with normal working stresses, and the accelerations of the reduced earthquake would be half those of the maximum earthquake.

In the above case, if the customary one-third increase is applied to the normal working stresses, use of spectra having ordinates two-thirds those of the maximum earthquake would be appropriate. For purposes of this paper the earthquake identified with the working stress basis will be termed the "reduced earthquake".

The working stress basis has practical advantages in enabling the use of allowable stresses and procedures which are familiar to the designer. However, this approach also has conservatisms and unconservatisms leading to non-uniform safety factors when the system performance is evaluated under the conditions of the maximum earthquake. Part of this arises from the fact that safety factors vary with the type of material used (reinforced concrete vs structural steel, for example) and with the design practices of the engineering discipline involved (structural vs mechanical engineering practice, for example). Thus, the working stress for reinforcing steel is usually 50% of the specified minimum yield stress; for structural steel the value is about 60%. Working stresses in piping are usually based on several criteria, including a percentage of the ultimate strength of the material, a percentage of the yield point, or on creep considerations. The use of materials without a well defined yield point requires that the yield point be arbitrarily defined.

Aside from these variations, others arise whenever forces, in addition to those of seismic origin, must be accounted for in the design. When a structure which has been designed for the reduced earthquake using conventional working stresses is checked to determine the effect of the maximum earthquake, the result may indicate a reserve strength exceeding minimum requirements or, conversely, a deficiency may be indicated. The first case occurs when the stresses due to non-seismic forces are additive to those due to ground motion as, for example, where significant gravity loads exist in a member simultaneously stressed by earthquake forces.

Figure 6 provides a schematic illustration. At point A the stress is equal to the gravity stress f_{dl}. At B the allowable working stress, f_r, is reached under gravity load plus the seismic effect of the reduced earthquake. If the seismic effect is increased to the value corresponding to the maximum earthquake (point C), the total stress, f_m, is less than the stress, f_{me}, permitted under conditions of the maximum earthquake. In this figure it is implied that the reduced earthquake is scaled down from the maximum earthquake in the ratio f_r/f_{me}.

The second case (deficiency in reserve strength) can occur when the effect of the non-seismic forces relieves the effect of the forces due to ground motion as, for example, where the gravity load righting moment is used to resist a part of the seismic overturning moment. This case is illustrated in Fig. 7 where the dead load moment, M_{dl}, is adequate to give a net righting moment, M_r, under conditions of the reduced earthquake, but a negative margin of safety would occur under the maximum earthquake due to the net overturning moment, M_m. Usually this would not happen unless the spectra of the reduced earthquake are less than two-thirds those of the maximum earthquake since conventional overturning safety factors normally are not less than 1.5.

The effects just described are linear but the total effect is not proportional to the intensity of seismic ground motion. There are, in addition, non-linear effects. One of these is the seismically induced wave action in fluid containers ("sloshing"). Here doubling the seismic input will more than double the computed wave height. If it is important to prevent spillage in open containers, it should be realized that freeboard requirements based on the reduced earthquake will be less than those based on the maximum earthquake because of the non-linear variation of wave height with earthquake motion intensity.

ENERGY ABSORPTION

Survivability under an earthquake loading depends on energy absorbing capability. When response is elastic, this ability has primarily to do with the ability to store the energy input of the earthquake as strain energy in the structure. When response is inelastic, this capability is primarily associated with the ability to dissipate the energy input through plastic deformations.

Even though critical elements may be designed on the assumption that yield strains will not be exceeded, it is very desirable to insure that ample capacity for energy dissipation exists in the event that inelastic action should happen. This requires that brittle modes of possible failure be eliminated and discourages the use of brittle materials such as unreinforced masonry, unreinforced concrete and cast iron in critical areas.

It also encourages design procedures which avoid possible premature non-ductile failures under overload conditions imposed by the maximum earthquake, such as shear and bond failures in reinforced concrete, and failures in steel, such as those due to local buckling. For structures of these materials, the necessary criteria and design rules for avoiding such problems are readily available in the provisions of the American Concrete Institute for ultimate strength design (Ref. 7) and in the rules for plastic design in steel of the American Institute for Steel Construction, and from other sources (Refs. 5, 8, 9, 10 and 11).

The greatest promise for economy in design of earthquake resistant structures appears to stem from approaches which consider energy dissipation due to non-elastic strains (Refs. 4, 12 and 13). This technique can be applied with confidence in certain simple cases if care and conservatism are used. This requires special attention to such considerations as the selection of an appropriate safety factor, connection details, and the secondary effects of inelastic deflections. Generalization of the approach to include more complex cases may require further research.

STRESSES IN PIPING

Operating conditions imposed on piping systems induce stresses from a variety of causes and require consideration of the effects of stresses due to restraints against thermal expansion and stresses due to pressure, gravity loading, earthquake, and possibly operational shock and vibration. In addition, the effects of fatigue, corrosion, creep and radiation embrittlement may have to be considered. Hence, allowable stresses, both as to magnitude and method of evaluation, differ from those used for structural steel.

In piping systems it is recognized that plastic strains may occur when the system is first put into service and that, if properly designed, further significant plastic strains will not occur under continued use. The criterion governing this behavior is that the amount of plastic flow, e_{pf}, should not exceed the yield strain, e_{yc}, of the material in the cold (minimum) condition plus the yield strain, e_{yh}, of the material in the hot (operating) condition as indicated in Eq. (1) of Fig. 8.

For design purposes this criterion is usually expressed in terms of stresses, based on the assumption that the yield stress or stress at impending plastic flow is some factor, n, times the working stress. This is the basis for the stress criteria of the American Standard Code for Pressure Piping. This criteria defines the lower limit of the maximum stress range between the cold condition and the hot condition to which a system could be subjected without undergoing plastic flow or yielding in flexure at either limit.

According to Markl (Ref. 14) this limit is conservatively estimated by using $n = 1.6$, which leads to Eq. (2) of Fig. 8, wherein S_c and S_h are the code allowable stresses at the minimum and maximum temperatures, (from Ref. 15) and S_{pf} is the stress range at which plastic flow or yielding in flexure is imminent.

For normal loading conditions the longitudinal stresses, S_e, S_p, and S_w, due to thermal expansion, pressure and weight, respectively, must meet the criterion of Eq. (3) of Fig. 8 which implies a lower limit of the safety factor under normal operating conditions of about 1.28 when the cyclic reduction factor, f, is unity. In Eq. (3) the sum of S_p and S_w must not exceed S_h.

In a design which satisfies Eq. (3), with f taken as unity, and where the sum of S_p and S_w is equal to S_h, the seismic stress increment, S_s, can equal the value shown in Eq. (4) of Fig. 8, without yielding, according to the criterion of Eq. (2).

DIFFERENTIAL MOTION

Differential motion can occur from relative ground displacements or from out of phase oscillations of responding components or combinations of these two motions, and can be an important consideration. Examples of such conditions include ducts extending between structures which are mounted on separate footings, and piping runs connecting flexible equipment items mounted on a common base or on separate footings. In such situations it may be necessary to estimate the maximum relative ground displacements, maximum amplitude of the oscillations of responding elements, or both.

In these cases the connecting elements between the displaced components must be capable of resisting the total displacement without loss of function. In general, this requires connections of flexibility sufficient to accommodate the total motion. In some cases it may be possible to use rigid connections, although the resulting system is likely to be more complex, dynamically, than a system with flexible connections.

DAMPING FACTORS

The spectrum method requires that the amount of damping be considered. The proper amount of damping to be used, and whether the assumption of viscous damping is appropriate, are two controversial facets of a subject wherein little quantitative data is available. Unfortunately, also, where the amount of damping is small, variations in the assumed amount of damping cause large variations in the design forces obtained from the spectra. Tentatively acceptable values of the percentage of critical damping for design are presented in Fig. 9.

These values imply elastic response. Damping factors increase as stress intensities increase. Even a small amount of yielding can be equivalent in energy absorption to elastic response with damping factors higher than those shown.

DATA ON PERIODS OF VIBRATION

Useful approximate formulas for estimating periods of vibration of structural components are contained in Refs. 16, 17, 18 and 19. The formula in Fig. 10 applying to straight runs of pipe, empty or water-filled, may also be of interest. Supports at each end are assumed to be non-deflecting and rigidly connected to the ground. Dimensions are in inches. Effect of rotational end restraints other than hinged and fixed can be approximated by linear interpolation or, more accurately, by formulas given in Ref. 17. Obviously, these formulas are not adequate to cope with complex one or two plane systems and are not intended for such use.

MINIMIZATION OF SEISMIC FORCES

A general observation which is perhaps obvious, but nevertheless sometimes useful, is that many of the reactor structures and equipment components have short periods of vibration — below 0.20 seconds or less. This condition differs from that found in tall buildings, where periods are significantly longer.

If the conventionalized spectra previously referred to are used as a design basis, it is found that the highest seismic forces result with natural periods of the structure in the range of 0.2 to 0.5 seconds. Consequently, the situation is approximately as shown in Fig. 11 by the solid curve. It is apparent that, in this case, seismic forces are minimized for short-period structures by stiffening the structure to further reduce the period, whereas for most conventional structures the reverse is true.

On the other hand, if the shape of the spectrum in region A is that shown by the dashed line, there is less to be gained by stiffening the structure, and minimization of seismic forces may be a real possibility only by reducing the stiffness, addition of damping, shock isolation, or other means.

IDEALIZED CLASSIFICATION OF STRUCTURES

A classification of structures according to response characteristics is helpful since response of a given component may fall typically in a certain category identifying the analysis approach required.

Rigid Body Structures

Thus, a massive structure or component may be so rigid that its fundamental period of vibration approaches zero. When founded on firm ground it may be permissible to assume that the structure moves as the earth moves and hence, is subjected to essentially the same peak horizontal acceleration as that of the ground. This condition is approached when the fundamental period of vibration is below a value in the range of 0.05 to 0.10 seconds for structures with moderate damping, and leads to appreciable simplification in determining seismic response.

Certain nuclear reactor structures typically fall in this category. Examples include most containment vessels and reinforced concrete shear wall type structures of low height to width ratio.

Earthquake forces in these so-called rigid body structures can be computed by simply applying, at any level, a horizontal force equal to the mass tributary to that level times the ground acceleration. In checking to determine whether the fundamental period of a structure is low enough to justify the rigid body assumption, it is important to recognize that shear deformations in structures having small ratios of span to effective depth may be more significant than deformations due to flexure.

Structures With One Degree of Freedom

The idealization as a single degree of freedom system fits a variety of structures where the effective mass of the structure can be considered to be lumped at a single point with acceptable error. Included are such structures as single story buildings, single-level equipment supports, and certain types of ground-supported fluid containers. Here, unlike the case of rigid body structures, the earthquake effect depends on the period of vibration of the structure and its damping, but is readily evaluated once these quantities have been estimated.

Structures With Two Degrees of Freedom

Examples of structures commonly idealized as two degree-of-freedom systems include two-story structures and fluid containers on elevated supports. For these cases evaluation of the earthquake effect is more complex and involves the determination of mode shapes, participation factors, and periods of vibration for two modes.

Multi-Degree of Freedom Structures

The complexity of many components requires idealization as a system with more than two degrees of freedom. Multi-story buildings and complex piping systems with associated equipment may require such treatment. In the case of piping systems the uniformly distributed mass of the piping is often lumped at discrete points, to give a multi-degree of freedom system. The most expeditious way of evaluating earthquake effects on such systems may be through the use of electronic computers. Programs capable of handling this problem are available and have been used for seismic analysis of reactor piping systems.

Systems with two or more degrees of freedom can also result when a flexible equipment item is mounted on a base which also has significant flexibility. In these cases, the base vibration differs from that of the earthquake ground motion. Since what the equipment "feels" is the base vibration, its own response may be considerably different from that obtained by assuming that the equipment is directly coupled to the ground. Such a case might occur, for example, when flexible equipment is mounted on an upper floor of a flexible structure.

Elastic Body Structures

So-called "elastic body" structures, wherein the distribution of mass is essentially continuous, such as chimneys and stand-pipes, represent a limiting case of multi-mass systems in which the number of masses becomes infinite. Where both mass and flexural stiffness per unit length are uniform, a fairly simple analysis of earthquake response can be made using available tabular data (Refs. 20 and 21). Where these quantities are not uniform, the structure can be idealized as a multi-mass system.

Examples of the calculations involved in most of the cases enumerated are given in Ref. 22.

SEISMIC CRITERIA USED AT THE HANFORD WORKS

Seismic criteria derived from dynamic considerations have been used in connection with reactor facilities at the Hanford Works in the state of Washington in the review of the earthquake resistance of certain features of existing production reactors, and also in design of a new reactor facility (Refs. 23, 24, 25 and 26).

These facilities are located in an area currently rated as zone 2 in the Uniform Building Code (Fig. 2). The following material briefly summarizes the main features of the seismic regulations applied to the recently designed NPR reactor at this site.

The principal intent of this criteria is to prevent malfunction of critical components and structures — features whose failure could cause either reactor runaway or a meltdown accident. Critical components include the reactor core, shields, and piping and supports essential to the flow of emergency coolant, including the water supply and pumping equipment. For such items the spectra of Fig. 12 were applied. These are versions of the conventionalized spectra of Fig. 4, scaled to a 20%g ground acceleration, and intended for use with ordinary working stresses increased the usual 33%. Thus, these are reduced spectra. They can be regarded as applying to a reduced earthquake approximately equivalent to the use of a maximum earthquake with a ground acceleration of 25%g. This acceleration was considered to be near the upper limit of the Mercalli VIII earthquake intensity chosen to represent the maximum earthquake. One application of the spectra consisted in checking the adequacy of critical piping systems through a computerized dynamic analysis.

The main reactor building of reinforced concrete was designed in accordance with the zone 3 requirements of the 1955 Uniform Building Code, the basic 13.3%g value therein being increased to 16%g, and other specified lateral force coefficients being increased, generally, in the same proportion with the usual one-third increase in allowable stresses permitted. The use of static coefficients for the reactor building was shown to be justified by preliminary studies of the building and its parts, which indicated in general, extremely short periods of vibration attributable to the shear wall type configuration of massive, short span construction (Ref. 24).

PROPOSED JAPANESE REGULATIONS

The Japanese have had under consideration tentative regulations (Ref. 27) for earthquake resistant design of nuclear power plants. Principal provisions are briefly summarized in the following material.

The regulations classify features in three categories according to function:

1. Buildings and civil engineering structures.
2. Mechanical structures, machinery and equipment.
3. Piping systems.

and in three classes, A, B and C, according to seriousness of the radiation threat implied by failure or malfunction. The most restrictive of the three main classes (Class A) requires elastic response under the ground motion of the so-called "strongest" earthquake, and the use of dynamic analysis, at least in principle. Class A components for mechanical and piping systems are further divided into subclasses. Some of the main features of these proposed regulations are summarized in Fig. 13.

In the case of buildings in Class A, the design may be made using a static analysis with a seismic coefficient of 3C where C is the basic seismic coefficient specified by the building code. The design is checked by a dynamic analysis based on the accelerations imposed by the strongest earthquake. A 50% increase in normal working stress, f_w, is permitted which, for structural steel, would result in yield point stresses. Thus, for cases wherein the building code coefficient of 20%g would apply for conventional structures, the static analysis for a Class A reactor structure would be based on 60%g.

Critical mechanical structures and piping (Class A) are designed to resist the forces based on an earthquake 1.5 times as intense as the intensity, P_m, used in the design of the Class A buildings with the design being based on the results of a dynamic analysis. Assuming that the basic allowable stress (S) is 25% of the ultimate tensile stress, the stress permissible under seismic conditions is 1.3S. This may be increased to 1.5S when thermal effects are considered. A 25% increase in these factors is allowed for purely structural parts of mechanical features, such as supports.

The proposed regulations also require seismic detectors and shutdown systems, and vibration tests of critical mechanical features whose dynamic behavior could not be evaluated during design.

EXAMPLES OF DYNAMIC ANALYSIS

The spectrum technique and dynamic methods have been applied in the design and review of numerous structures typical of reactor installations (Refs. 22, 23, 24, 25 and 26). The following material describes the results of a few of these investigations. While the structures selected are not peculiar solely to reactors, they were chosen because they are simple enough structurally to be directly investigated without involving a large number of simplifying assumptions requiring explanation. In each case described, the earthquake input consisted of the average spectra for ground motion having an intensity equal to that of the 1940 El Centro earthquake as shown in Fig. 4.

300-Foot Chimney

One of these cases involved the review of a 300-foot reinforced concrete chimney used as an exhaust stack for a nuclear reactor. As far as is known, earthquake considerations did not enter into the original design. The dimensions of the structure are shown in Fig. 14. The outside diameter tapers linearly from 22 feet 5 inches at the bottom to 12 feet at the top. The maximum wall thickness is 14 inches at the base, decreasing to a minimum of 6 inches at 187 feet above the base. Above this level the thickness remains constant at 6 inches. Reinforcing steel specified consists of intermediate grade deformed bars conforming to ASTM A 15, with 40 diameter minimum laps at splices. The vertical reinforcing steel, located near the outside surface, varies from 7/8 inch bars near the bottom of the shaft to 1/2 inch bars near the top. Ring steel consists of 3/8 inch bars at 6 inches located near the outside face. Concrete strength used in design is 3000 psi. The chimney contains a 4-inch perforated radial firebrick lining terminating at about 113 feet above the base. The lining is in three sections of approximately equal length, and is supported on corbels, the upper two sections being separated from the concrete walls by an air gap. In the lower section the gap is filled with 2 inch fiberglass insulation. The principal structural discontinuity in the chimney shell is a pair of flue openings located about 40 feet above the top of the footing. The openings are about 5 feet 6 inches wide by 17 feet high and they pierce the chimney wall at locations 90° apart on the chimney periphery. The chimney footing is octagonal, 7 feet thick and has a base width of 34 feet across flats. The stem of the footing is 25 feet across flats and about 3 feet high. The footing pour is specified to be monolithic, from which it is inferred that the usual construction joint between the base and stem is omitted. Anchorage of the chimney shell to the footing utilizes 102 1-inch dowels. These dowels project through the stem and into the footing base about 3 inches, terminating in a 90° hook. Concrete strength in the footing is specified as 2500 psi and the allowable soil pressure is 8000 psf.

The dynamic analysis consisted of evaluating influence coefficients by determining the deflected shape of the chimney for single unit loads applied successively at each of the ten division points of the chimney where the tributary mass was assumed to be concentrated. From these deflections a set of equations was evolved which stated that the deflection of each mass is the deflection due to the inertia force acting on the given mass plus that due to the inertia forces acting on all the other masses. Solution of the determinant of the set of equations yielded the mode shapes and periods of vibration for the first three modes through a standard eigen-vector eigen-value analysis performed on an electronic computer. These mode shapes and periods of vibration, T, are shown in Fig. 15. The values for the first two modes agree quite well with check values obtained from a manual solution utilizing Stodola's method. From this data normalized modal shears, normalized modal bending moments and participation factors, K, were obtained as shown in Figs. 16 and 17. The actual deflections, shears and bending moments in each mode, based on an assumed damping of 15% critical, were calculated using the normalized modal values and the participation factors. The absolute sum of the modal bending moments is shown in Fig. 18. This envelope can be considered to represent an upper bound of the bending moment at any point in the chimney. This can be compared in the figure with the moment capacity of the chimney calculated on an assumption of a 30,000 psi steel stress. It is apparent that overstress exists on this basis in the middle part of the chimney. The maximum moment at the base obtained from this envelope is equivalent to that provided by a static lateral force of about 16%g.

It is often argued that at the base of chimneys the effect of modes higher than the first mode tends to be overstated. It is also apparent that the simultaneous occurrence of maxima in each mode is largely a matter of chance. Consequently, the designer will often attempt to reduce the envelope values to something less on a more or less rational basis. Such an envelope is indicated by the curve labeled "Modified Envelope" in Fig. 18. This envelope is obtained by taking the square root of the sum of the squares of the first three modes as has been proposed by Goodman, Newmark and Rosenbleuth (Refs. 28 and 29). It is seen that the modified envelope falls below the moment capacity curve throughout the entire chimney except for some overstress in the vicinity of the flue openings. The fourth curve in this figure shows the moments obtained using the approach described by Housner in Ref. 30, modified to allow the direct use of the spectra of Fig. 4 in lieu of the approximate analytical expression for the spectra derived in Ref. 30. This curve is based on 20% damping. It is seen that it compares well, generally speaking, with the modified envelope except near the upper portions of the chimney.

The base moment from a 25 pound per square foot wind is also indicated in the figure. Although not evident from the figure, it appears that the wind condition is more critical than the seismic condition in the lower portion of the chimney. This is due to the fact that in a typical wind design in accordance with the ACI chimney code, (Ref. 31), the allowable stresses, which are much lower than those used here, result in a more conservative design in spite of the fact that the wind bending moments are generally lower.

The seismic integrity of the brick lining and inadequate anchorage of reinforcing steel in the stem of the footing would constitute other problem areas in this structure in the event of an earthquake of the assumed intensity.

Wind-Braced Water Tower

The following discussion summarizes results of a review aimed at estimating the seismic resistance of a wind-braced water tower used in supplying coolant to a reactor. The principal dimensions of the tower are shown in Fig. 19. The total height is about 150 feet. The tower shaft, with sloping legs, is hexagonal in plan form with a 14 WF-119 pound leg at each corner. The tank, of 300,000 gallon capacity, is cylindrical, with elliptical heads at top and bottom, and is connected to a 5-foot diameter riser having a 3/8 inch wall thickness. The riser is stayed at bracing levels by 3/4 inch diameter radial rods attached to each of the six legs. Wind bracing consists of rods ranging in diameter from

1-1/4 inches in the upper panels to 1-1/2 inches in the lower panels. Horizontal struts consist of pairs of channels, one flat, the other vertical. This tower was investigated from the standpoint of behavior as an elastic structure and also under deformation in the plastic range as obtained from a so-called limit analysis. Damping values of 2% critical and 0.5% critical were used for the tower motion and for the fluid motion, respectively.

For the elastic response calculation, a tensile yield stress of 33 ksi was used and the strength of compression members was assumed to be 1.8 times the basic value obtained from the conventional AISC column formula. For the limit analysis the tensile yield stress of 33 ksi was retained. Strengths of compression members and strength of members under combined axial thrust and flexure were computed according to Ref. 32.

The statically applied horizontal force which could be applied to the tower as governed by the value of elastic capacity of the members is shown in Fig. 20. It is seen that the critical members are the diagonals in the upper panel, which limit the statically applied horizontal force to about 3%g at the onset of yielding.

The stress in the members of the tower due to a horizontal force at the top of the tower was obtained by making conventional assumptions and the deflection at the top was calculated by the method of virtual work. The horizontal force at the top of the tower required to produce a horizontal displacement of 1 foot was found to be about 420 kips.

The elastic analysis considered two modes of vibration of the tower. The first mode is largely sloshing of the fluid, whereas the second mode is primarily tower motion. The second mode contributes a major portion of the load in the tower shaft. The two-mass system representing the structure and contents was obtained by converting the mass of water and an assumed tributary portion of tower shaft and tank shell into two equivalent solid masses, both considered to be attached to the tank base, one rigidly and the other flexibly. Mode shapes, periods of vibration and participation factors were obtained for these two modes of vibration (Refs. 22 and 23) and results are summarized in Fig. 21. The first mode has a period of about 4 seconds and the second slightly more than 2 seconds. The modal shears shown are based on participation factors of about 1.5 for the first mode and 0.5 for the second mode. The absolute summation of these modal shears gives a maximum horizontal shear in the tower of about 414 kips, which corresponds to a lateral force factor of about 15%g. Under this loading, as anticipated, all members of the tower would be greatly overstressed and the footings and anchorage would also be inadequate to resist the uplift forces. As governed by the strength of the rod bracing, it was estimated that the tower could withstand, elastically, a ground motion of about 7%g.

It is evident that the use of heavier diagonal rods would be the most effective way of increasing the seismic resistance of the tower. However, increasing the rod strength would not proportionally increase the intensity of the seismic motion at which the tower response remains elastic, because strengthening the tower would decrease the period of vibration. This would have the unfavorable effect of increasing the horizontal force acting on the tower.

A limit analysis was performed to evaluate seismic capability of the tower in the yield range, by comparing the energy input of the earthquake with the energy dissipating capacity of the structure, based on the assumption that connections have the strength to develop the full capacity of the members to which they are attached. Conclusions from the limit analysis are summarized as follows:

1. Yielding of rods would cause a redistribution of shear between the various tower faces above level 4, leading to compressive loads in some of the radial rods of the horizontal bracing at levels 4 and 5. These loads could well exceed the initial tension in the rods and possibly could cause tower collapse due to instability.

2. A large displacement of a single panel due to rod stretch would be structurally intolerable in its effect on the riser, the horizontal bracing, and tower legs.

3. The critical leg in the top panel would be subjected to bending stresses about both principal axes due to the panel displacement which would limit severely the amount of plastic strain that could be permitted in the tower diagonals. Unfortunately, an accurate estimate of leg capacity in this skewed bending condition was handicapped by a lack of adequate engineering data.

Earthquake Braced Tower

The following discussion summarizes a review of an earthquake-resistant water tower used in supplying coolant to a reactor. Details are shown in Fig. 22. The tank capacity of this tower was about 300,000 gallons. The total height of the tower was about 25 feet greater than that of the wind-braced tower. The analysis of this tower indicated that the upper limit of elastic response would be obtained at a ground motion intensity of about 19%g. Deficiencies in the footing might result in uplift at intensities of about 22%g — an intensity substantially less than that required to develop the yield strength of the diagonals in the top panel.

A limit analysis was also performed, ignoring the deficiencies at the base of the tower. Results of the limit analysis indicated that the amount of energy to be dissipated by stretching of the diagonals in the upper panel would require a 2 to 3 inch maximum transient horizontal displacement of the top level with respect to the level below in order to resist a ground motion of 33%g intensity. Of this panel displacement, about 1 inch would be elastic, leaving a permanent set of less than 2 inches. Aside from the deficiency in uplift previously mentioned, this tower is obviously a much better earthquake risk than the wind-braced tower previously discussed and would require a relatively small amount of rework to insure survival under a 33%g ground motion.

The results of these and other tower investigations point to the need for assessing the strength of the weakest element and providing elements and connections with sufficient capacity throughout the structure to develop the strength of the weakest link. This requires an analysis based on conditions in the plastic range as a supplement to the conventional working stress approach.

CONCLUDING REMARKS

The earthquake resistant design of nuclear reactors is a challenge which transcends the boundaries between engineering disciplines in the sense that structural engineering considerations become primary factors in regard to mechanical and electrical features. In the absence of a backlog of seismic performance history this creates unusual demands on the designer. However, complex systems have been successfully designed to survive shock inputs many times greater than that of earthquakes as, for example, in "hardened" missile facilities and nuclear submarines. This fact indicates that, while the design of nuclear power reactors with a high degree of seismic resistance does require an extremely thorough coordinated engineering effort, the desired result is within the capability of our present-day technology.

REFERENCES

1. Uniform Building Code, 1961 Edition, Volume 1, International Conference of Building Officials.

2. C. F. Richter, Seismic Regionalization, California Institute of Technology, Seismological Laboratory, Division of Geological Sciences, Contribution No. 897, December 1958.

3. G. W. Housner, Behavior of Structures During Earthquakes, Proceedings of the American Society of Civil Engineers, Journal of the Engineering Mechanics Division, Volume 85, No. EM 4, October 1959.

4. J. A. Blume, Structural Dynamics in Earthquake-Resistant Design, Transactions of the American Society of Civil Engineers, Vol. 125, 1960.

5. J. A. Blume, N. M. Newmark, L. H. Corning, Design of Multistory Reinforced Concrete Buildings for Earthquake Motions, published by Portland Cement Association, 33 West Grand Avenue, Chicago 10, Illinois, 1961.

6. W. K. Cloud, Maximum Accelerations During Earthquakes, presented at the Chilean Meeting on Seismology and Earthquake Engineering, July 15-19, 1963.

7. Building Code Requirements for Reinforced Concrete, (ACI 318-63), American Concrete Institute, June 1963.

8. Manual of Steel Construction, Sixth Edition, American Institute of Steel Construction, 1963.

9. Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, American Institute of Steel Construction, Adopted April 17, 1963.

10. *Commentary on the Specification for the Design, Fabrication and Erection of Structural Steel for Buildings*, American Institute of Steel Construction, Adopted April 17, 1963.

11. *Commentary on Plastic Design in Steel*, ASCE Manual of Engineering Practice No. 41, American Society of Civil Engineers.

12. G. W. Housner, *Limit Design of Structures to Resist Earthquakes*, Proceedings of the 1956 World Conference on Earthquake Engineering, Earthquake Engineering Research Institute, San Francisco, 1956.

13. G. W. Housner, *The Plastic Failure of Frames During Earthquakes*, Proceedings of the Second World Conference on Earthquake Engineering, Tokyo, Japan, 1960.

14. A. R. C. Markl, *Piping-Flexibility Analysis*, Transactions, American Society of Mechanical Engineers, February 1955.

15. American Standards Association, *Code for Pressure Piping*, ASA B 31.1-1955.

16. *Design of Structures to Resist Nuclear Weapons Effects*, ASCE Manual No. 42, American Society of Civil Engineers, 1961.

17. *Design Manual - AEC Test Structures*, prepared and published by Holmes & Narver, Inc. for U.S. Atomic Energy Commission, December 1961.

18. R. A. Williamson, *Performance and Design of Special Purpose Blast Resistant Structures*, Journal of the American Concrete Institute, May 1960.

19. J. N. MacDuff and R. P. Felgar, Jr., *Vibration Design Charts*, Paper No. 56-A-75, American Society of Mechanical Engineers.

20. D. Young and R.P. Felgar, Jr., Tables of Characteristic Functions Representing Normal Modes of Vibration of a Beam, University of Texas Publication No. 4913, University of Texas, Austin, Texas, July 1, 1949.

21. R.P. Felgar, Jr., Formulas for Integrals Containing Characteristic Functions of a Vibrating Beam, Circular No. 14, Bureau of Engineering Research, University of Texas, Austin, Texas.

22. Reactors and Earthquakes, U.S. Atomic Energy Commission.

23. Earthquake Resistance of Hanford Production Reactors, A Study for General Electric Company, Hanford Atomic Products Operation, Richland, Washington, Contract No. AEC-AT(04-3)-174, Project Agreement No. 2PO 18-123, Report HN-146, by Holmes & Narver, Inc., June 1960 (Classified).

24. Earthquake Resistance of the Hanford New Production Reactor, A Study for General Electric Company, Hanford Atomic Products Operation, Richland, Washington, Contract No. AEC-AT(04-3)-174, Project Agreement No. 2PO 18-123, Report HN-148, by Holmes & Narver, Inc., June 1960.

25. Hanford New Production Reactor-Structural Review of 105-N Building, for General Electric Company, Hanford Atomic Products Operation, Report HN-156, by Holmes & Narver, Inc., July 1961.

26. Hanford New Production Reactor - 105-N Building - Engineering Analysis of the Inlet Connector Support Structure and the Inlet Thermal Barrier, for General Electric Company, Hanford Atomic Products Operation, Report HN-173-8044, Volumes I and II, by Holmes & Narver, Inc., December 1962.

27. K. Takeyama, Earthquake Resistant Design for Nuclear Power Plants in Japan, Proceedings of the Symposium on Reactor Safety and Hazards Evaluation Techniques held in Vienna, May 14-18, 1962, sponsored by the International Atomic Energy Agency.

28. L.E. Goodman, E. Rosenbleuth, N.M. Newmark, Aseismic Design of Firmly Founded Elastic Structures, Transactions, American Society of Civil Engineers, Vol. 120, 1955.

29. E. Rosenbleuth, Some Applications of Probability Theory in Aseismic Design, Proceedings, World Conference on Earthquake Engineering, Earthquake Engineering Research Institute, Berkeley, California, 1956.

30. G.W. Housner, Earthquake Resistant Design Based on Dynamic Properties of Earthquakes, Journal of the American Concrete Institute, July 1956.

31. Specification for the Design and Construction of Reinforced Concrete Chimneys, (ACI 505-54), American Concrete Institute, 1954.

32. Plastic Design in Steel, American Institute of Steel Construction, 1959.

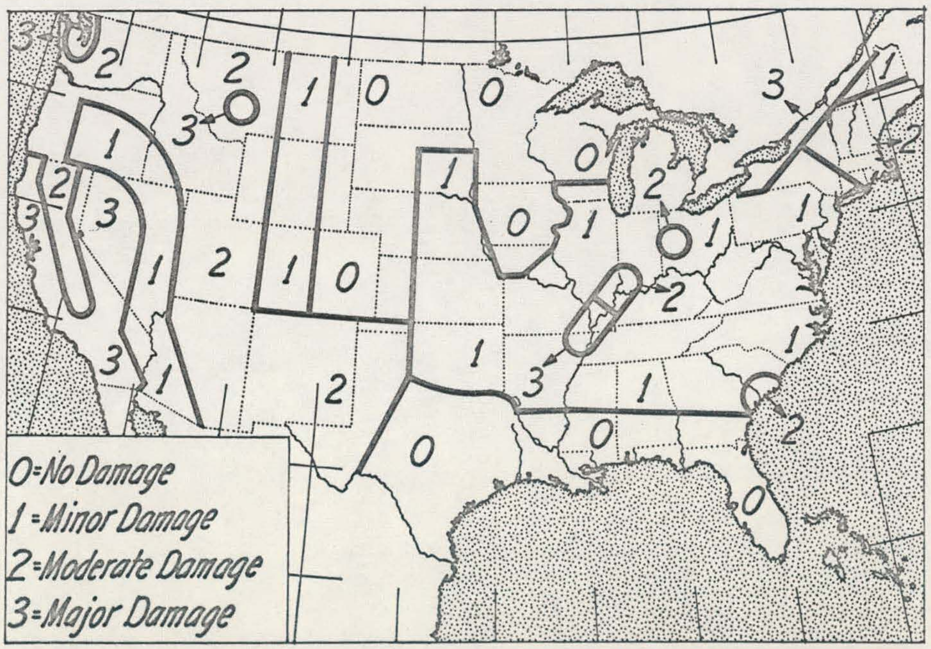
33. G.W. Housner, The Dynamic Behavior of Water Tanks, Bulletin of the Seismological Society of America, February 1963.

34. G.W. Housner, Design of Nuclear Power Reactors Against Earthquakes, Proceedings of the Second World Conference on Earthquake Engineering, Tokyo, Japan, 1960.

1. Reactor Internals
2. Coolant System
3. "Poison" Injection System
4. Containment Vessels
5. Biological and Thermal Shields
6. Fuel Storage Pool
7. Control Room
8. Sources of Coolant, Power and Fuel
9. Ventilation Systems

CRITICAL REACTOR COMPONENTS

FIGURE 1



USA SEISMIC PROBABILITY MAP USC&GS-UBC

FIGURE 2

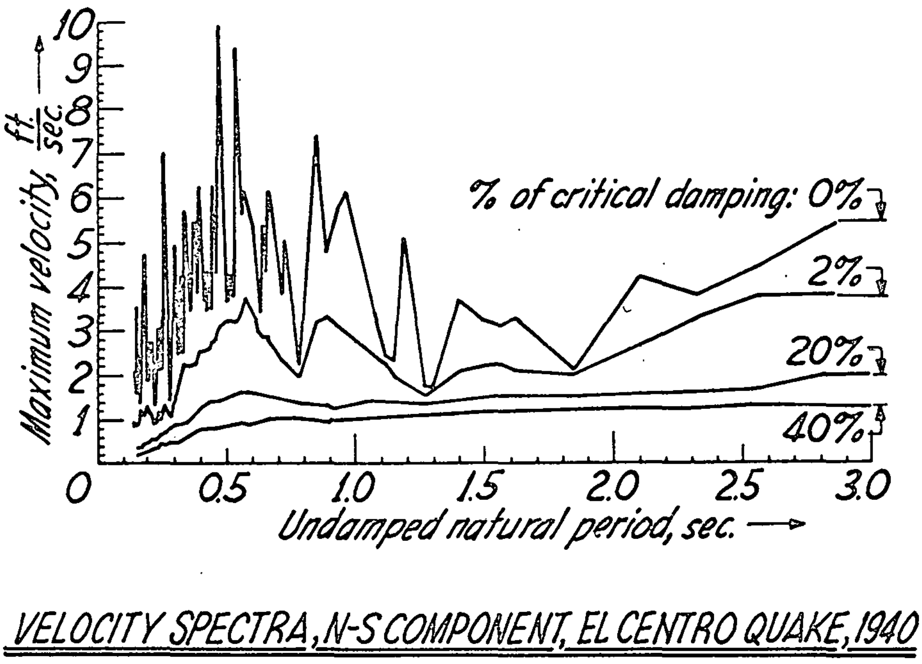


FIGURE 3

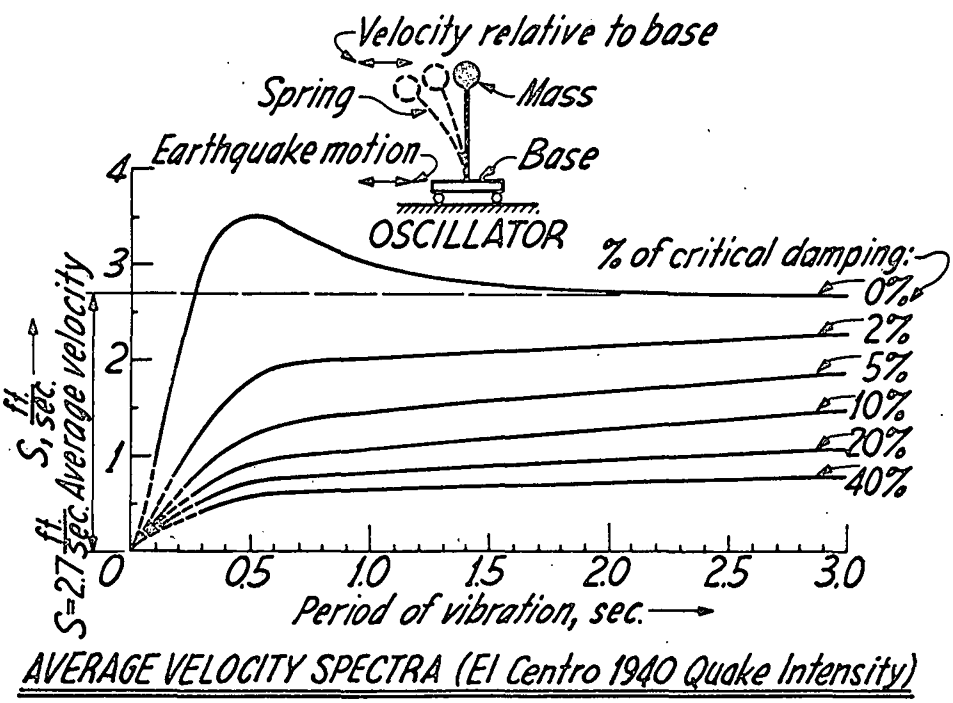


FIGURE 4

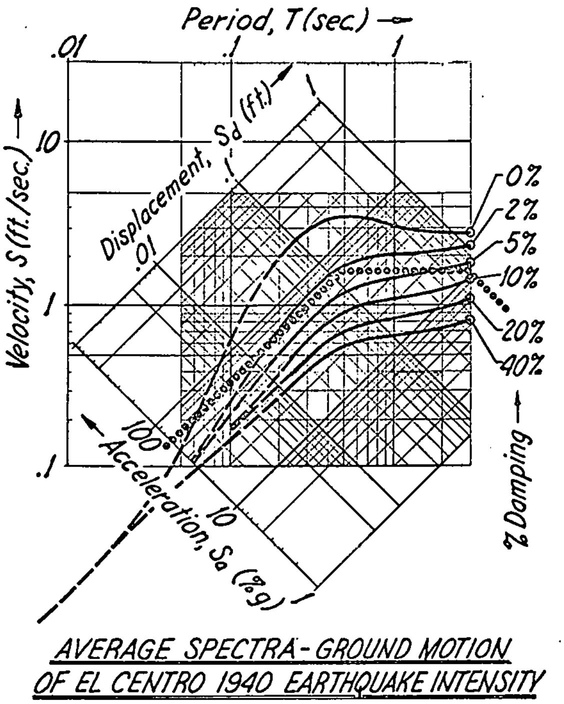


FIGURE 5

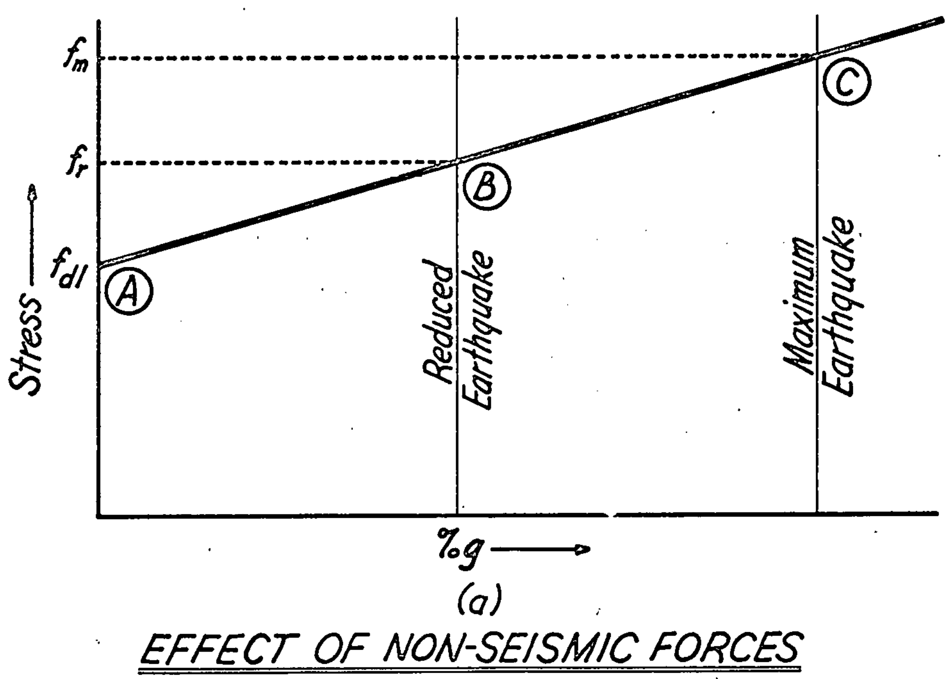
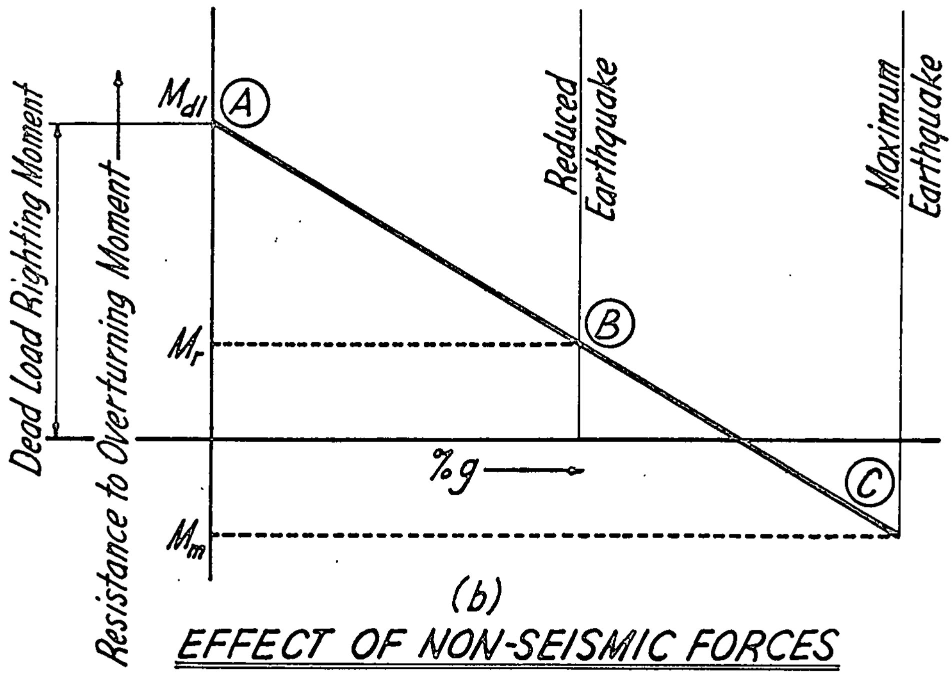


FIGURE 6



EFFECT OF NON-SEISMIC FORCES

FIGURE 7

$$e_{pf} = e_{yc} + e_{yh} \text{ ---------------- Eq.(1)}$$

$$S_{pf} = 1.6\,(S_c + S_h) \text{ -------------- Eq.(2)}$$

$$S_e + S_p + S_w \geq 1.25 f\,(S_c + S_h) \text{ ------- Eq.(3)}$$

When $f = 1$ and $S_p + S_w = S_h$, at yield,

$$S_s = 0.35\,(S_c + S_h) \text{ -------------- Eq.(4)}$$

STRESSES IN PIPING

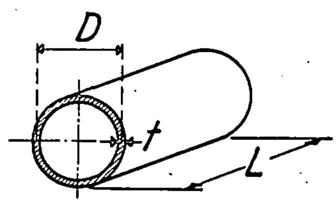
FIGURE 8

Type of Structure		% of Critical Damping
Unfireproofed Steel Structures	Welded	0.5 to 2
	Riveted	2 to 5
Concrete		5
Masonry		15 to 40
"Sloshing" motion in fluid containers		0 to 0.5

TENTATIVE DAMPING FACTORS FOR ELASTIC RESPONSE

FIGURE 9

$$T \approx \frac{L^2}{D}\left(\frac{C_1}{C_2}\right)$$

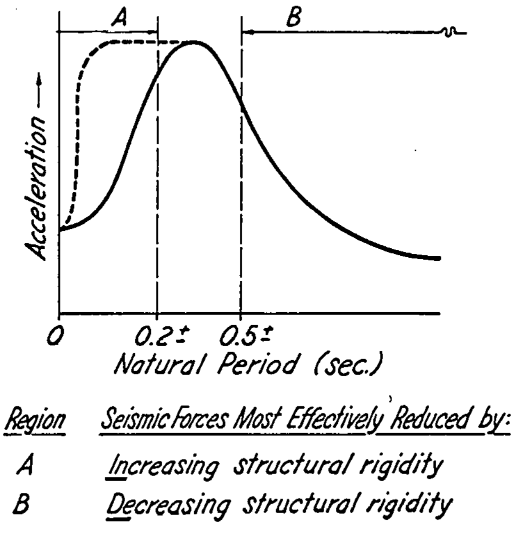


$$C_1 \begin{cases} \text{Pipe Empty} \text{.............} 1 \\ \text{Pipe Full} \text{.........} \sqrt{1+0.0319\,\frac{D}{t}} \end{cases}$$

$$C_2 \begin{cases} \text{Hinged Ends} \text{..........} 113{,}200 \\ \text{Fixed Ends} \text{..........} 256{,}900 \end{cases}$$

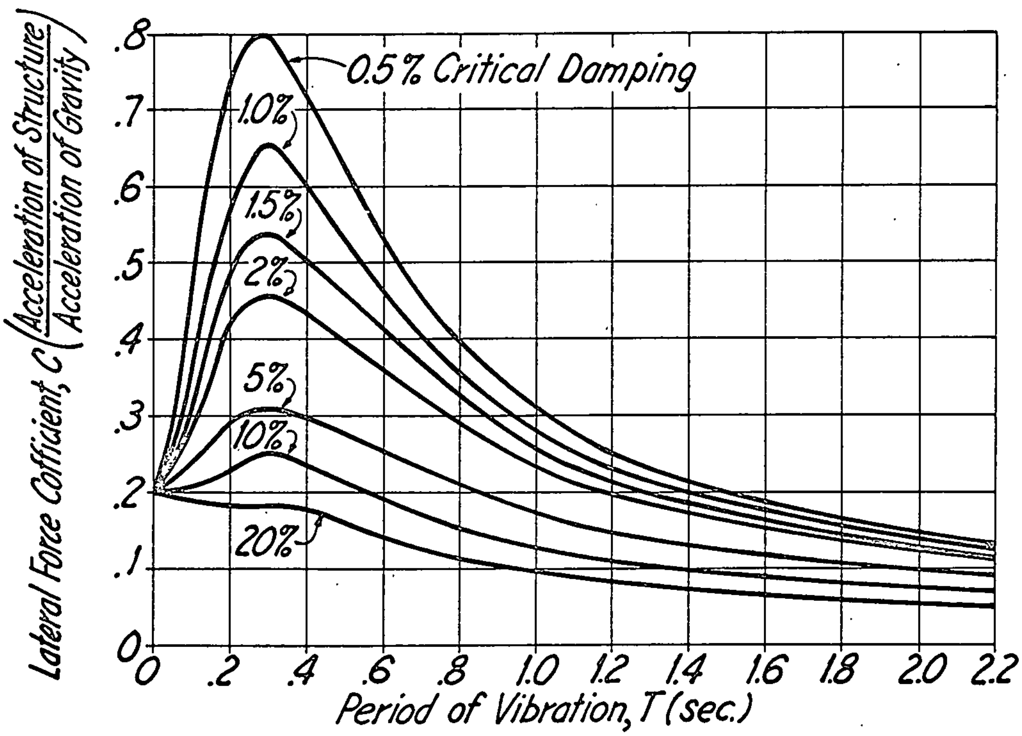
PERIODS OF VIBRATION FOR STEEL PIPE

FIGURE 10



SEISMIC FORCES VS. STRUCTURAL RIGIDITY

FIGURE 11



PROPOSED SPECTRA FOR DESIGN OF HANFORD REACTOR FACILITIES

FIGURE 12

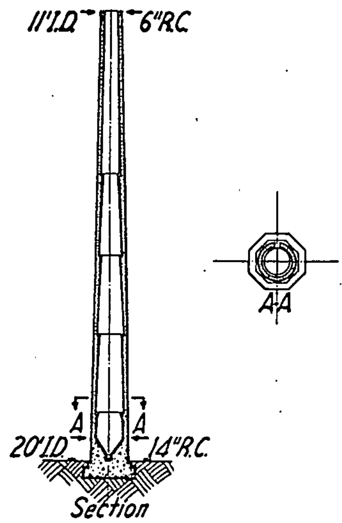
Class A Structures

Static Analysis	Dynamic Analysis		Allowable Stresses Under Seismic Conditions	
Buildings, enclosures, supporting structures				
3C	P_m		$1.5 f_w$	
Mechanical structures, equipment, piping				
——	Class A_1	Class A_2, A_3	Process Components	Structural Components
——	$1.5 P_m$	P_m	1.3S 1.5S*	1.63S 1.88S*

*Applicable when thermal effects are included.

PROPOSED CRITERIA FOR POWER REACTORS IN JAPAN

FIGURE 13



300' CHIMNEY

FIGURE 14

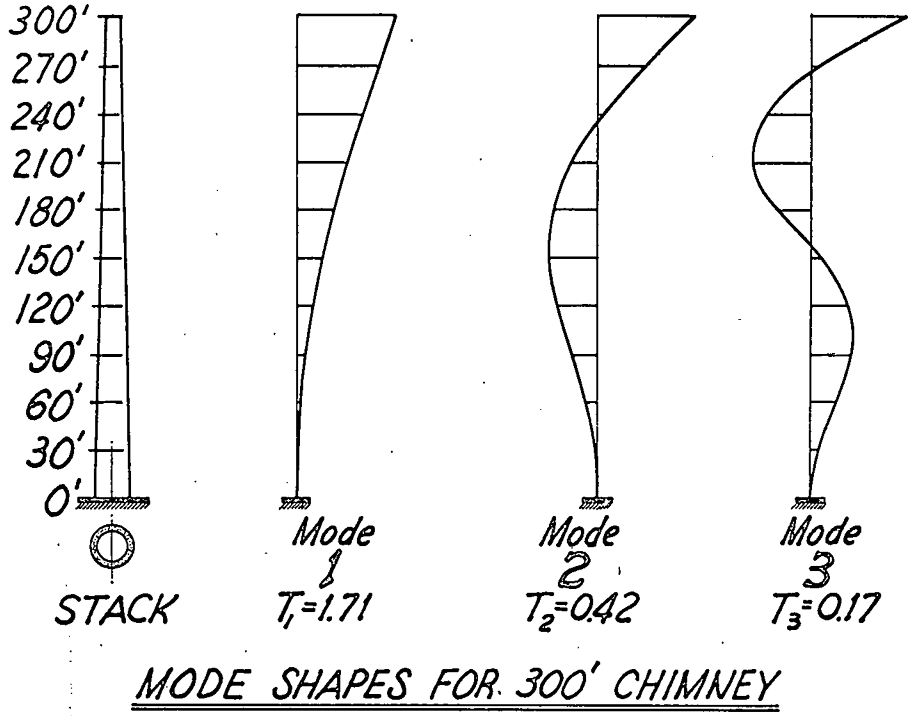


FIGURE 15

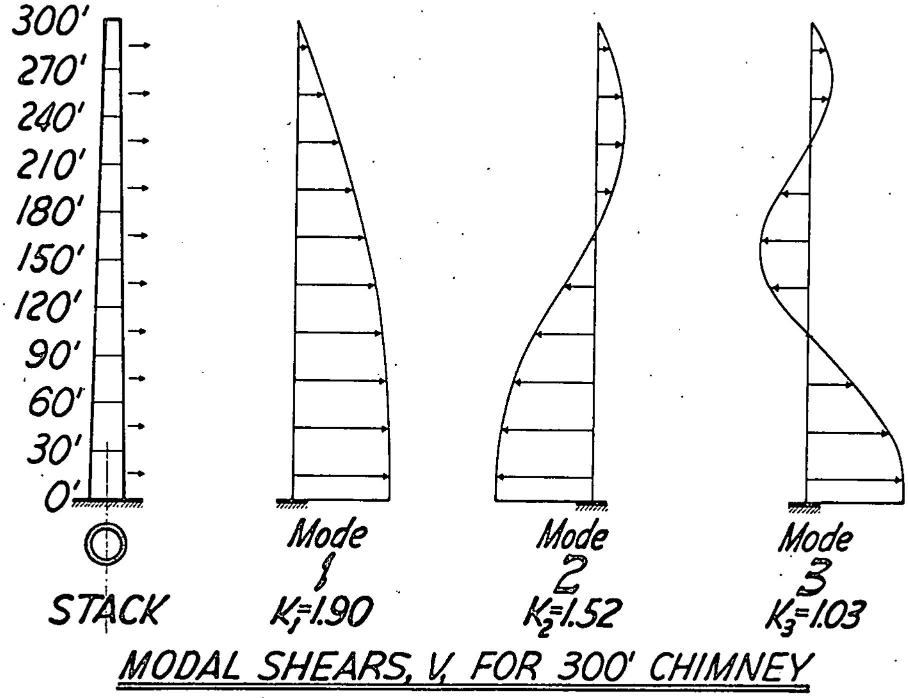


FIGURE 16

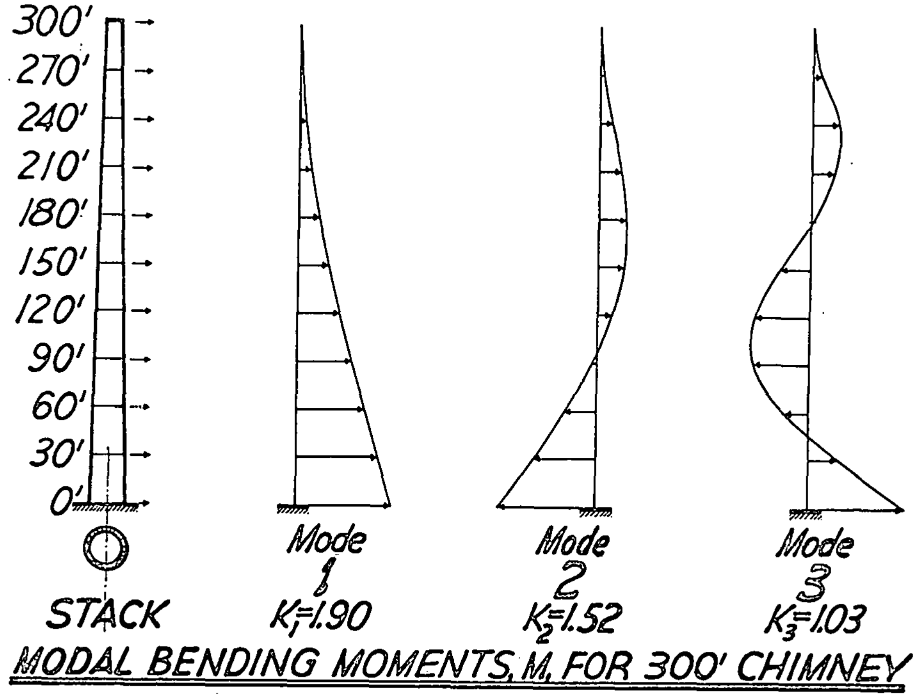


FIGURE 17

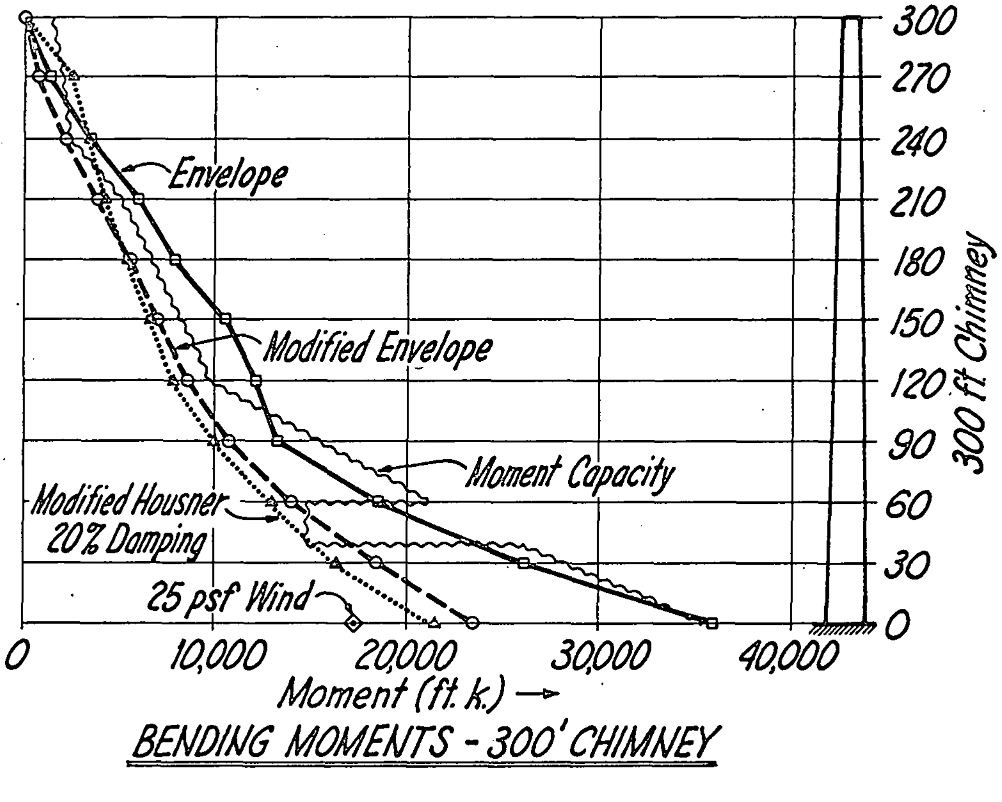


FIGURE 18

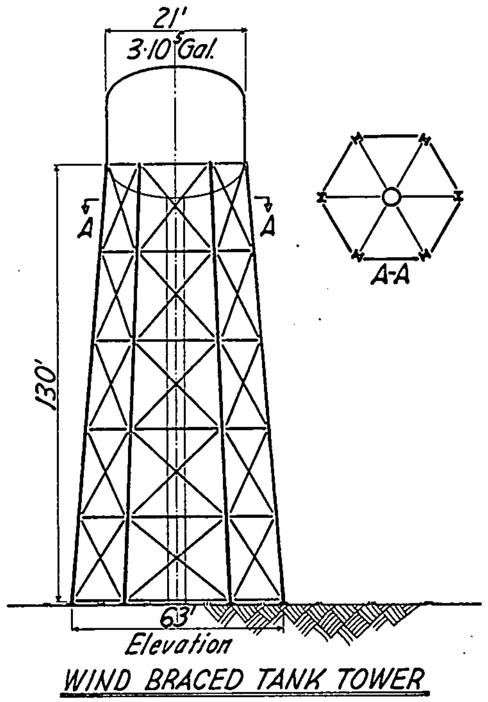


FIGURE 19

Tower Member	Upper Limit of Static Horizontal Shear (%g)
Diagonal (tension)	3
Strut (compression)	14
Leg (compression)	12
Footing (uplift)	11

ELASTIC CAPACITY OF CRITICAL TOWER MEMBERS

FIGURE 20

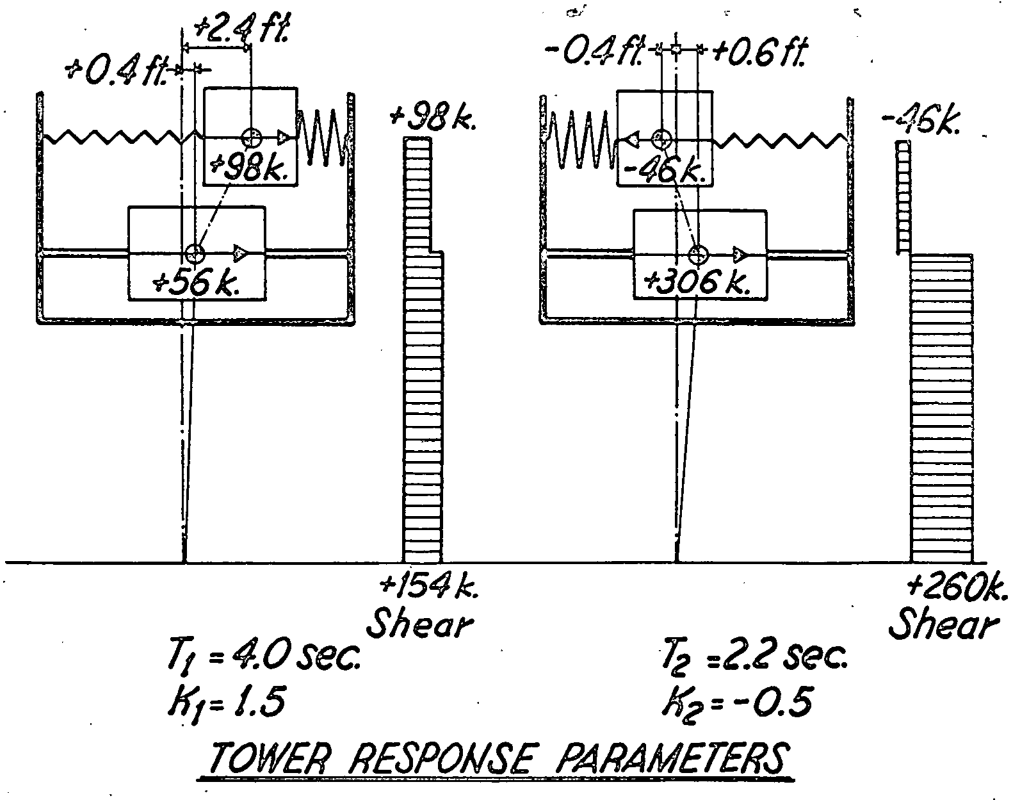


FIGURE 21

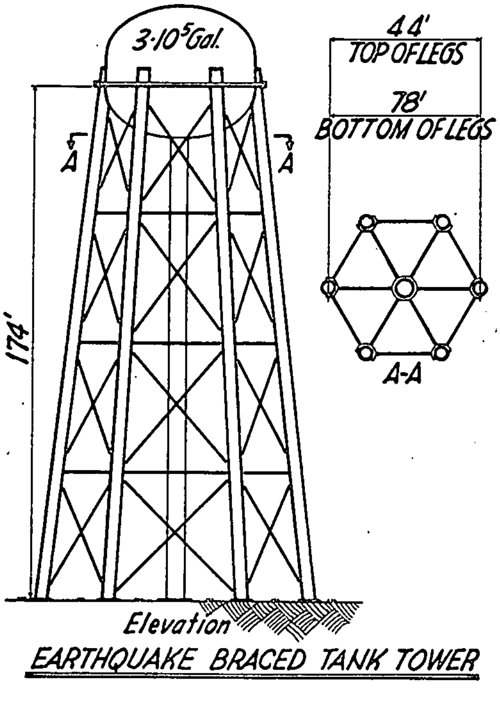


FIGURE 22