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STRUCTURAL DESIGN CONSIDERATIONS AND SAFETY AGAINST EARTHQUAKES

BODEGA BAY ATOMIC PARK UNIT NO. 1

by

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June 25, 1963

1. INTRODUCTION

This review is based on consideration of the material contained in References 1 through 6, listed at the end of this report. In addition, an estimate has been made of the maximum credible earthquake intensity that may be experienced at the site, based on information from References 7 and 8. Although the estimate used herein for the maximum credible intensity differs appreciably from that used in the application, (see especially Reference 3), if one takes into account the design stresses, reasonable values of energy absorption, etc., then for the appropriate choice of damping factors the net effect on the design obtained by the writer's analysis will not differ appreciably from the effect obtained using the procedures suggested by Dr. Housner in Reference 3. However, it is important to specify precisely the energy absorption factor and the percentage of critical damping, as these affect the design parameters to a major degree.

Although it is entirely feasible to design the proposed reactor and the pertinent structures to resist the maximum credible earthquake intensity that might be experienced at the site, such a design will

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require careful attention to detail, and particularly to the possibility of large relative motions developing in the near vicinity, which must be taken into account in providing for power lines, pipe lines, and other means of communication between the reactor and points at or beyond the San Andreas fault.

It is a basic assumption of this report that the design will be made with methods consistent with the best available knowledge and information, including theoretical studies of earthquake resistant design, and will not be dependent solely on standard codes and specifications developed for buildings of entirely different configuration, in which the hazards of partial failures are not nearly so severe.

2. NATURE OF EARTHQUAKE MOTIONS

In an earthquake the ground moves in a more or less random fashion and in more or less random directions, although there is generally a preferential direction of motion parallel to a major fault plane. The vertical motions are definitely less than the horizontal motions in regions such as those near the San Andreas fault. Measurements have been made in recent decades by the U. S. Coast and Geodetic Survey of the "strong motion" accelerations, as a function of time, at a number of points on the West Coast of the United States. The record of strongest motion which has been obtained so far is that for the El Centro Earthquake of May 18, 1940. For this record, in the North-South direction the maximum ground acceleration was about 0.33 g; integration of the accelerogram gives values of maximum ground velocity of slightly less than 14 in. per sec., and maximum ground displacement of the order of slightly more than 8 in.

On page 1 of Reference 8, it is stated that "this is the most severe earthquake motion for which accurate records are now available; it may be considered as an earthquake to be expected in a specific location in California with an estimated frequency of once in 50 years, or more often if the region is close to more than one active fault. Somewhat larger motions would no doubt be experienced close to an epicenter."

The ground velocity and ground displacement, obtained by integration of the ground acceleration record, are shown in Fig. 1-1 on page 2 of Reference 8. It is of interest to examine the characteristics of the motion shown by this figure. The ground velocity is characterized by many fewer peaks with a relatively longer time between successive peaks than the ground acceleration. The ground displacement has only a limited number of peaks, with as much as five or six seconds between successive peaks of ground displacement. The total duration of large motions was of the order of 30 seconds for the El Centro Earthquake.

In general, large motions occur in steps or pulses adding up to the maximum value, but each step or pulse is characterized by having a relatively short time base. Consequently, the maximum displacement is not at all a measure of the severity of the earthquake in terms of its effect on a structure which moves as a unit with the ground on which it is supported. On the other hand, the maximum ground velocity is a reasonable measure of the energy imparted to the structure, and consequently the energy that must be absorbed by deformation of the structure. Hence, the maximum ground velocity is the best single measure of actual damaging or destructive tendency in an earthquake. For very brittle structures, the maximum acceleration of the ground is the best measure of the damaging tendency. This is generally true for structures of any characteristics whatsoever where their fundamental period of vibration is less than 0.1 or 0.2 sec.

The motions near the ground surface, or at the surface, on which a structure is founded arise from the large and violent displacements generally along a fault plane or zone, with the major disturbance originating at some depth of the order of 10 to 20 miles below the surface of the earth. Consequently, the accelerations and velocities near the surface trace of the epicentral fault are not substantially larger than those at some distance of the order of several miles away, although, the maximum permanent displacement might be substantially greater near the surface trace of the fault along which the major motion has taken place.

It is possible for a structure of relatively compact form, well tied together, to survive even by bridging across a surface fault. It will be moved bodily and subjected to large dynamic forces. On the other hand, a structure which has separate supporting elements bridging across a fault may be completely wrecked by the large movements of one side of the fault relative to the other. Mere proximity to the fault zone is not necessarily a measure of the damage potential to a structure, independent of the characteristics of the structural design.

3. INTENSITY OF SHOCK TO BE CONSIDERED

In Reference 7, the estimate is made that the maximum acceleration in the maximum credible earthquake at or near the site would be of the order of about twice that measured in the El Centro Earthquake record previously discussed. Earthquakes of intensity about equal to the El Centro record have occurred in the past, and have even exceeded it (notably the 1906 San Francisco Earthquake), and it can be expected that this intensity will almost certainly be exceeded in the future. Although it is difficult to estimate the strength of future earthquakes, there is evidence that it is possible for an intensity of the order of IX, on the so-called Modified Mercalli Intensity scale, to occur in or near the region of Bodega Bay. A crude estimate, based on the probabilities of occurrence of earthquakes of various intensities, leads to the conclusion that the maximum credible earthquake near a fault may have maximum ground accelerations, on soil or soft, loose rock, of the order of 0.5 to 0.7 g, maximum ground velocities of the order of 30 in. per sec., and maximum displacements of several feet. On bedrock the accelerations would be higher, (in the range from 0.75 to 1.0 g), but the displacements possibly lower with about the same maximum velocity. The maximum displacements at or in the fault zone may be as much as 5 to 10 feet, dropping rapidly, however, to values of the order of 3 to 5 feet at points definitely outside the fault zone, and to less than 2 to 3 feet at distances of the order of a mile or more away. However, the displacements are significant only insofar as relative motions between parts of a large structural complex are concerned.

With reasonable accuracy, the intensity of expected maximum motion, in terms of accelerations and velocities, may be taken as about twice that for the North-South direction of the El Centro Earthquake of May 18, 1940.

4. DESIGN TO RESIST EARTHQUAKE MOTIONS AND SHOCK

The response of a structure to earthquake shock motions is generally most conveniently determined by use of the "response spectrum". (See Chapter 1 of Reference 8.) The response spectrum is a plot, against period of vibration or natural frequency of vibration, of either the relative displacements or strains in the structure, the relative velocities (measuring the energy absorption within the structure), or the maximum accelerations of the components within the structure, and can be used in ways that have been described in many technical papers and reports to determine the behavior of a structure when subjected to an earthquake, or to determine the necessary strength of the structure to resist earthquake forces and motions.

Within the range of periods of vibration of importance in most structures or structural element, the response spectrum can be approximated reasonably well by three constant limits or bounds. These bounds are generally described as the bound for relative displacement within the structure, the bound for relative pseudo-velocity (or circular natural frequency times relative displacement), and the bound for maximum acceleration of the masses of the structure. For moderate amounts of damping, of the order of 10 to 20 percent critical, these bounds are as follows:

maximum relative displacement = 1.0 maximum ground displacement
maximum pseudo-velocity = 1.5 maximum ground velocity
maximum acceleration = 2.0 maximum ground acceleration.

For relatively small amounts of damping, of the order of 2 to 5 percent critical, these bounds are increased, to approximately the following values:

maximum relative displacement = 1.33 maximum ground displacement
maximum pseudo-velocity = 2.0 maximum ground velocity
maximum acceleration = 3.0 maximum ground acceleration.

For structures having a frequency of vibration in the fundamental mode of more than 1.5 to 3 cycles per second, the acceleration bound is the appropriate design parameter. For structures having a lower frequency, down to 0.1 cycle per second, the pseudo-velocity bound is the appropriate design parameter. Consequently, it would be only for extremely flexible and long-period structures, having a period of more than 10 seconds, where the displacement bound would be of importance, and hence it can be neglected in the futher considerations of this report.

The use of the above spectra in design involves a choice of the level of stress considered and the amount of plastic deformation permissible. Design at working stress rather than at yielding implies a factor of safety of the order of about 2, using the working stresses in the "Uniform Building Code", as a basis, (not increased by the factor of 1/3 for lateral loading, however). Hence a spectrum intended to be used at yielding should be divided by a factor of 2 in order to be consistent with a spectrum intended to be used at working stress levels.

A reduction in the design spectrum is possible if one is willing to allow yielding in the structure after an earthquake shock has occurred. The effect of inelastic behavior of structures is described in more detail in Chapters 1, 2 and 3 of Reference 8. However, in the following we shall be primarily concerned with structures designed to remain elastic in order to provide for a sufficient margin of safety under unusual conditions where partial failure or overstress might be extremely hazardous.

Since the basis for design described in Reference 4 involves design for normal working stresses, under the Uniform Building Code, in order for the recommended spectra herein to be comparable, the values described in Section 3 of this report must be divided by a factor of 2. When this is done, the spectrum in Reference 4 for 2.5 percent damping, lies reasonably close to the values obtained from the arguments herein, for structures founded on soil, but lies considerably below in the range of frequencies greater than 2 cycles per second, for structures founded on rock. For such structures the spectrum value given in Reference 4 for 2.5 percent damping corresponds to a maximum acceleration, for design purposes, of approximately 0.9 g, whereas the recommendations given herein would be between 1.1 and 1.5 g.

However, this is not a matter that is impossible to resolve. To increase the proposed design spectrum in the appropriate range of frequencies would not involve undue hardship or inconvenience.

The provision for differential motions within a structure is more complex. However, there are two factors that must be considered. Where different elements are founded on the same firm base but have connections between them, each of them can respond relative to the base with a response determined from the shock response spectrum. The relative displacement determined from the spectrum is the displacement of the mass of the structure using the base as a datum. If the elements have relatively small damping, their relative displacement (between the individual masses) can be the sum of the absolute maxima of their displacements relative to the base, since they may under certain conditions deflect out of phase with one another. Provision for such relative displacement must be made where the responding elements are connected together in any way by connections, piping, wiring, etc. This is true even if they are housed in rigid containers, but are free to displace or deform within that container.

The second type of differential displacement that must be considered is that corresponding to the different base motions that may be experienced by the different parts of the base of a structure. Where a large structure is not founded on a single integral base footing or mat, the individual parts can move relative to one another, both under transient conditions, and permanently. The magnitudes of these relative motions are difficult to determine, but they should be provided for through use of flexible connections, etc., so as to avoid tearing or damaging the vital parts of the structure because of the motions imposed on the different parts of the support. This condition is not one for which it is impossible to make adequate provisions in the design. However, it is not yet spelled out completely and in detail in the description of the proposed design.

Where flexible connections or piping with such connections are used between different parts or components in a complex structure, the joining or fastening to each of the parts must be considered to avoid tearing or rupture at these joints. Fastenings of piping to the reactor shell, and other connections of various kinds, must be designed to provide for the requisite forces. These may be larger than those corresponding to the accelerations for a mechanical or structural element supported directly on a foundation which moves with the intensity of ground motion described above. The reason for the increase is that the responding element to which the piping, etc., is fastened, has a motion of its own which may be larger than that of the foundation or base, and may, in fact, be more or less periodic, giving rise to much larger forces in elements supported by it. In other words, more sophisticated analyses of the system, taking into account the fact that the system is a multi-degree of freedom dynamical system, are required if the design is to avoid damage to component parts and elements connected to flexible or responding objects.

Provision must be made also for surges of water or fluid that may act upon submerged elements or nearly submerged elements of the structure.

Such surges, caused by the earthquake motions, produce forces on the components which may cause damage if they are not provided for in the design.

Finally, attention must be paid to the dynamic stability of earth or rock slopes in the vicinity where such slopes may, under the influence of earthquake motions, slide down and out in such a way as to cause large and unusual forces to be applied to structures, piping, cables, etc., in the path of the motion. Provision can be made to increase the static factor of safety of earth and rock slopes to the point where they will not be subject to failure under the dynamic conditions arising in an earthquake. The precise conditions are dependent on the properties of the material and the intensity of the earthquake motions, but in general factors of safety against sliding, under static conditions, of the order of 1.5 or more are required to prevent difficulties. Critical slopes, the failure of which might endanger either the structure or the utilities connected with the structure, should be examined to provide for an adequate factor of safety under dynamic conditions.

5. RECOMMENDATIONS AND COMMENTS

The conditions at the site do not appear to be unduly or unreasonably hazardous nor are the motions to be expected unduly or unreasonably large compared with those which have occurred in other regions of the world or even in the state of California. *Although the displacements and the velocities and accelerations may seem quite large, much larger shock motions are experienced in ships and submarines, and materially larger motions have been designed for in underground control centers and missile launching facilities.

It is entirely feasible to design the proposed reactor to resist the maximum credible earthquake shock at the site. To insure that the design will be adequate, however, requires further examination of a number of factors in detail, which can best be done during the course of the design itself. A dynamic analysis of the final structure, as designed, should be made to assure that adequate provisions have been made for relative motions, etc. The provision in Reference 4, Part 3, that "in addition to the foregoing elastic design, a further analysis will be made to insure that ground motion five times as intense as the design spectrum will be required to produce incipient failure of structures", is more than adequate to give a reasonable assurance of appropriate strength for the structure, if the analysis is adequately made. The analysis should provide for consideration of relative motions, however, as well as for stresses within the structure.

Special attention should be paid to the effects of the earthquake accelerations on brittle and critical elements of the reactor itself, including the fuel rods and the control rods. A dynamic analysis of these parts of the structure, taking into account the motions of the container in which they are placed, as excited by the earthquake motions of the ground, should be made.

6. REFERENCES

1. Preliminary Hazards Summary Report, Bodega Bay Atomic Park Unit Number 1, submitted by Pacific Gas and Electric Company, Dec. 28, 1962, (Docket No. 50-205), See especially Chapter V, Section D.

2. Report on Earthquake Hazards at the Bodega Bay Power Plant Site, by Don Tocher and William Quaide, Appendix IV to Reference 1.

3. Earthquake Hazards and Earthquake Resistant Design - Bodega Bay Power Plant Site, by George W. Housner, Appendix V to Reference 1.

4. Amendment No. 3 to Reference 1, Part III.

5. Amendment No. 1 to Reference 1.

6. Amendment No. 2 to Reference 1.

7. Report by Frank Neumann.

8. Design of Multistory Reinforced Concrete Buildings for Earthquake Motions, by John A. Blume, Nathan M. Newmark, and Leo H. Corning, published by Portland Cement Association, Chicago, 1961.

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